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# A comparative study of measured and computed stresses in the Thirteenth street bridge over Skunk River near Ames, Iowa

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# A COMPARATIVE STUDY OF MEASURED AND COMPUTED STRESSES IN THE THIRTEENTH STREET BRIDGE OVER SKUNK RIVER NEAR AMES IOWA

BY

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## ILLUSTRATIONS AND TABLES

FIG.I. VIEWS OF BRIDGE TAKEN DURING THIS INVESTIGATION.

## INTRODUCTION

Opportunity. Last fall as school opened, Mr. Sigmund Eiken and the author, were presented with the opportunity of measuring the stresses in a highway bridge, about to be erected, near Ames Iowa. This structure is located just east of Ames, on Thirteenth Street extended, and spans the Skunk River at this point.

Organization and Acknowledgment. Since the bridge was being erected at the time we started work, we found it necessary to press into service; two senior civil engineering students, who were familiar with this type of work. The author wishes to take this opportunity of expressing his hearty appreciation of the services rendered by Mr. Albert J. Anderson and Mr. George E. Lamp in connection with this project. In addition he wishes to thank the Iowa Engineering Experiment Station and the American Society of Civil Engineers for the loan of men, equipment and instruments; and the contractors, Mr. Ben Cole and The Pittsburg Des Moines Steel Company, for their co-operation and assistance.

In view of the fact that the time available for strain gage measurements was limited to the erection period, we found it necessary to use two parties for this work. Mr. Anderson took the readings for the first party with Mr. Lamp or who-ever was available as note-keeper, and the author took the readings for the second party with Mr. Eiken as note-keeper. This arrangement was followed thro-out all of the work.

The Bridge. As previously stated this bridge is located near Ames Iowa, and is a 120-foot span, 20-foot roadway, through riveted steel highway bridge, with an 8-inch concrete floor on steel stringers. A view of the bridge is shown in Fig. 1 and elevation of the truss in Fig. 2. The bridge was designed under the Iowa State Highway Commission Specifications of 1919 and is a T-type, through riveted Pratt truss.

Object of this Investigation. In planning the work it was thot that it would be possible to make a complete study of the action of the bridge during erection. This would have resulted in the measurement of the erection stresses, the stresses due to the weight of the steel and the stresses due to the weight of the concrete floor. Conditions and lack of time caused the scope of the work, to be reduced to the measurements of the floor load stresses. These stress measurements were taken on all of the members at the two upper and two lower panel points at the east end of the north truss of the bridge. From these readings we obtained the primary and secondary stresses in the structure, and a comparison of the observed and computed values will be made. In this connection a study of the distribution of stress in the upper chords and end post was also deemed advisable.

A complete study of the observed and computed stresses in the transverse bents was also made.

In addition the action of the portal members was studied for floor load stresses and stresses due to a horizontal load, applied to represent the action of the wind.

In the following work a plus sign will be used to designate tensile stresses while a minus sign will be used to designate compressive stresses.

## DISCUSSION OF STRAIN GAGE WORK

Instruments. Two 8-inch West strain gages were used for taking all of the readings. One of these instruments was the property of the College, while the other was loaned for the work by Prof. Almon H. Fuller. No time will be taken in explaining the operation of these instruments, as we may assume that all persons interested in this report, are familiar with this type of instrument.

Standard Bars. Two types of standards were used in this work. That used by Mr. Anderson was made from a $\frac{3}{4}$-inch gas pipe of wrought iron or soft steel of low carbon content, properly capped at the ends and provided with a thermometer inside. That used by the author was a section of 6-inch I-beam of structural steel. Since readings were taken before the sun came up in the morning or on cloudy days, the temperature range encountered in the investigation was from 40 degrees F. to 60 degrees F. With this slight difference in temperatures, it does not seem possible that the differences in the materials in the structure and standards, would be enough to cause an appreciable error in the results of this investigation. No readings were taken when the temperature was rapidly changing.

Location of Gage Points. Fig. 2 is an elevation of the north truss of the bridge showing the sections at which readings were taken. Letters are used in designating the

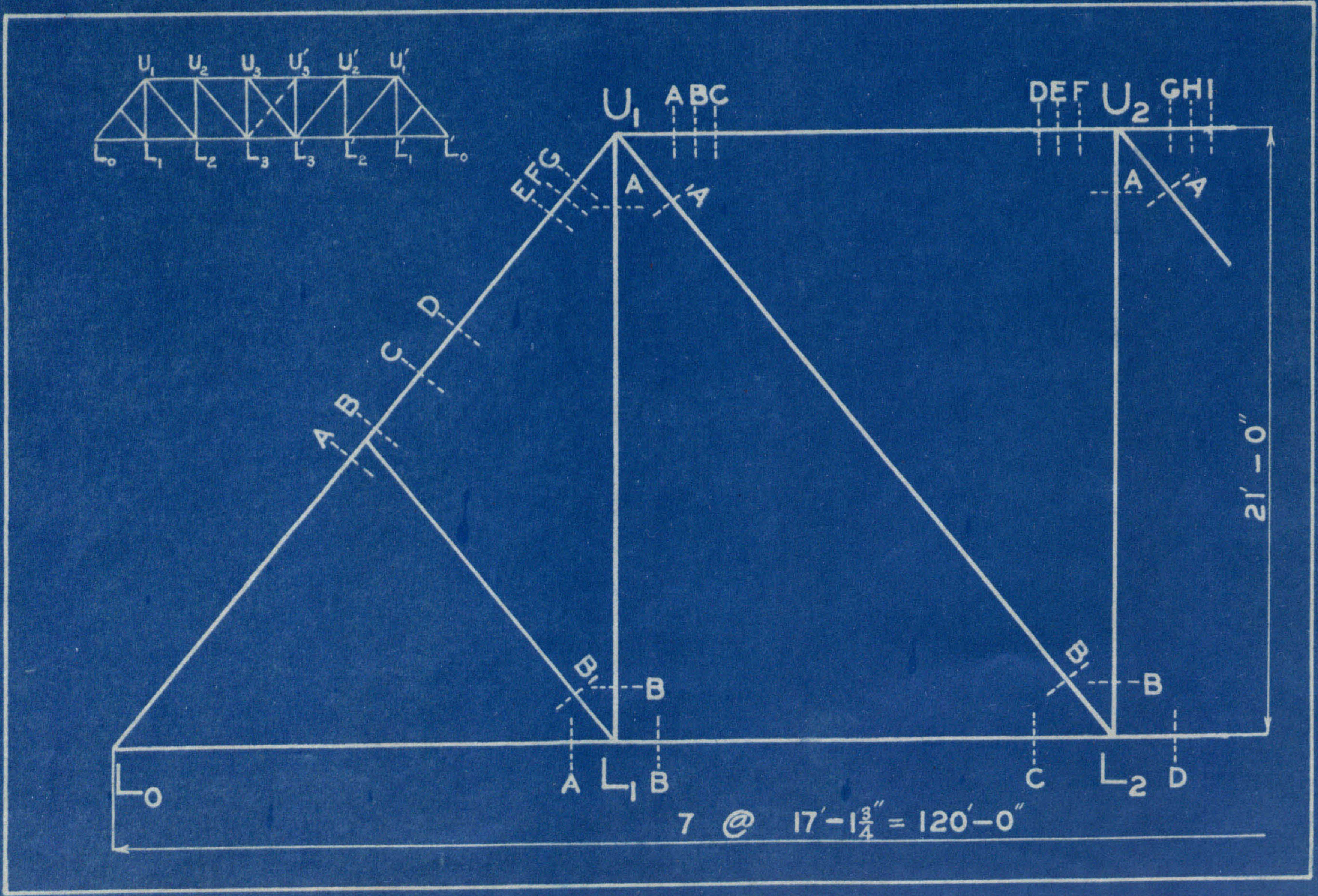



FIG. 2. ELEVATION OF TRUSS SHOWING LOCATION OF GAGE LINES

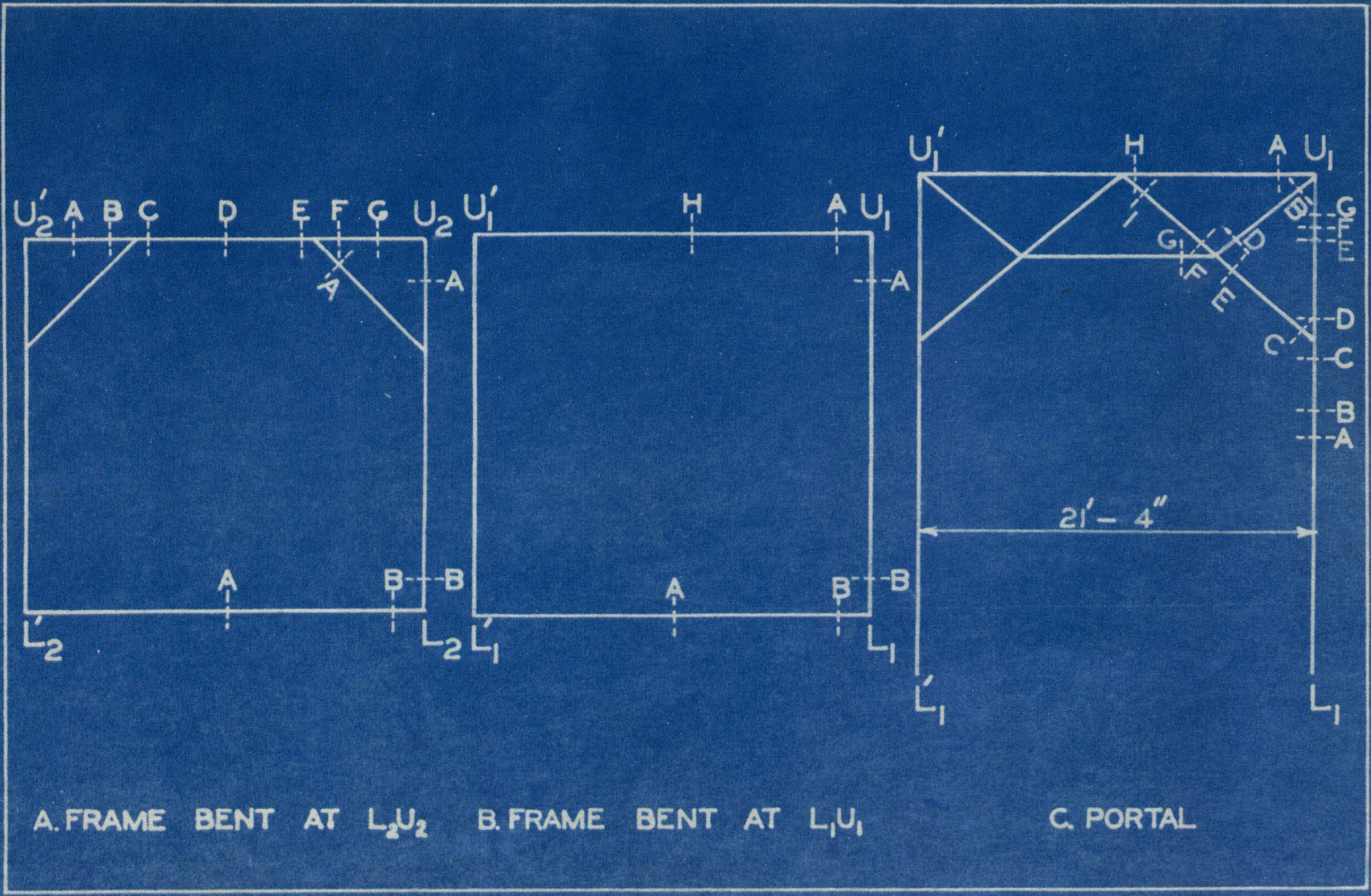



FIG. 3. ELEVATION OF TRANSVERSE BENTS SHOWING LOCATION OF GAGE LINES

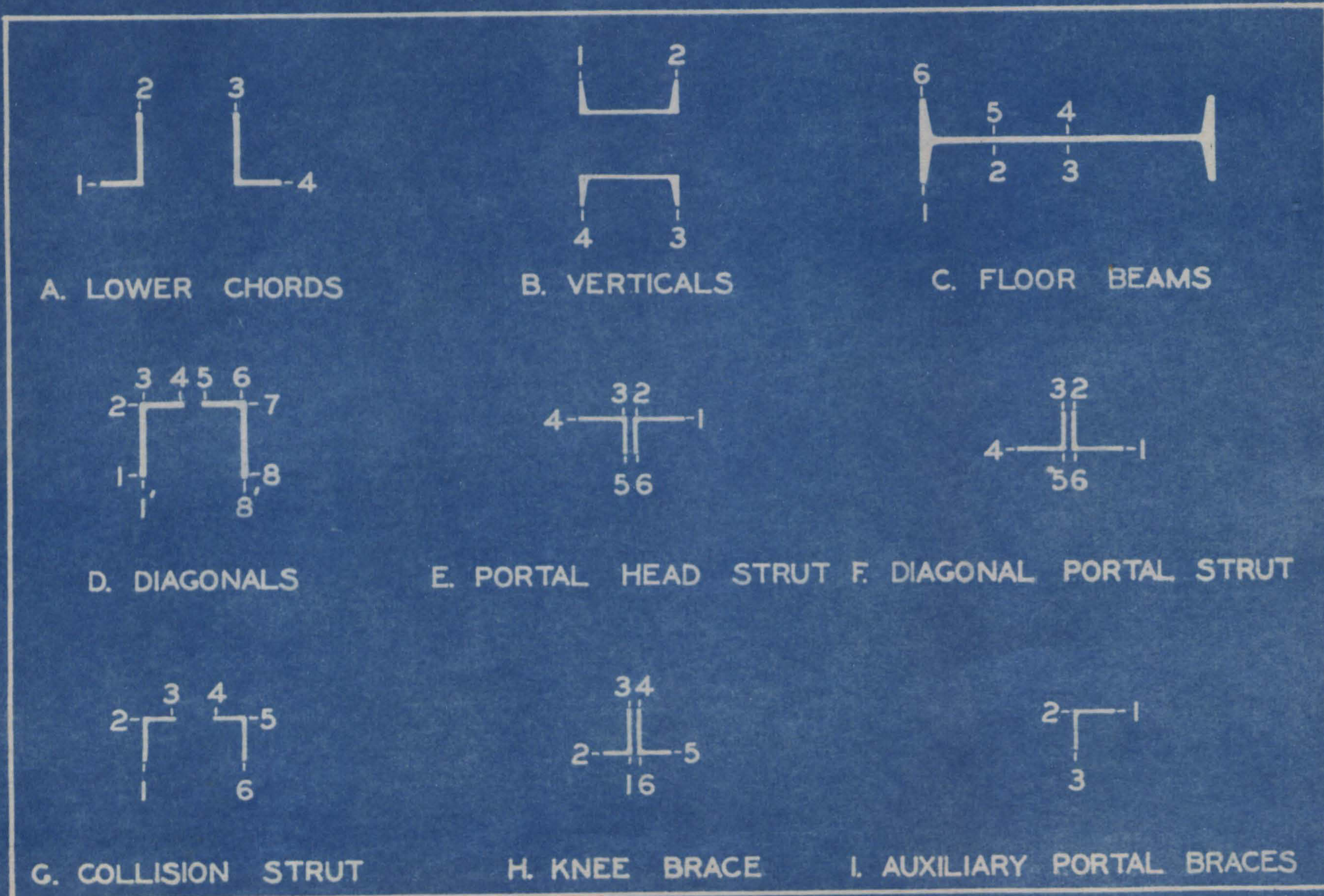



FIG. 4. LOCATION OF GAGE LINES ON VARIOUS MEMBERS

sections, while the gage lines are numbered consecutively about the members. This same system was used in the case of the members of the transverse bents and portal, and Fig. 3 shows the sections at which readings were taken on these members. The sheets following this discussion give the location of the various sections with respect to the nearest working point. Fig. 4 shows the methods used in numbering the various gage lines on the different members of the bridge.

Strain Gage Readings. When work on this project started, the lower chord members, floor beams, verticals and diagonals were already in position, supported by the centering, and so the gage holes were drilled and preliminary readings taken with these members in this position. The remaining members of the bridge were still on the ground and so the remaining gage holes were drilled and preliminary readings taken before they were moved into position. At this point the plan was to take readings during erection to determine the erection stresses, but the work of erecting the bridge progressed so rapidly, that we found it impossible to do this. A feature which kept us from obtaining a good set of readings before the bridge was swung, was the manner in which the centering had been erected. It seemed that no very great effort was made to have the tops of the bents in the same plane, and so most of the panel points were at different elevations. This caused bending in the members and any movement or settling of the piles, which formed the centering, caused our initial readings to behave in a most erratic manner. For this reason all of the readings taken while the bridge was supported by the centering, were disregarded in the following discussion.

After the bridge was swung, complete sets of readings were taken on all gage lines, before the riveting of connections began. Another complete set of readings were taken immediately after the riveting had been completed. These sets of readings checked very closely, which indicates that the riveting had no noticeable effect upon the stresses in the members.

The next and last set of strain gage readings were taken after the concrete floor had been poured and the forms removed.

## LOCATION OF GAGE LINES ON VARIOUS MEMBERS.

### End Post

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Section | A | is | 0' - 10" | below | collision | strut | connection. | |
| " | B | " | 0' - 6½" | above | " | " | " | |
| " | C | " | 0' - 11" | below | portal | " | " | |
| " | D | " | 1' - 1" | above | " | " | " | |
| " | E | " | 2' - 2" | below | working | point | at | U1. |
| " | F | " | 2' - 10" | " | " | " | " | " |
| " | G | " | 3' - 6" | " | " | " | " | " |

### Upper Chord

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Section | A | is | 1' - 11" | west | of | working | point | at | U1. |
| " | B | " | 2' - 7" | " | " | " | " | " | " |
| " | C | " | 3' - 3" | " | " | " | " | " | " |
| " | D | " | 1' - 3½" | east | " | " | " | " | U2. |
| " | E | " | 1' - 11" | " | " | " | " | " | " |
| " | F | " | 2' - 7½" | " | " | " | " | " | " |
| " | G | " | 1' - 10" | west | " | " | " | " | " |
| " | H | " | 2' - 6½" | " | " | " | " | " | " |
| " | I | " | 3' - 2½" | " | " | " | " | " | " |

### Lower Chord

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Section | A | is | 1' - 6½" | east | of | working | point | at | L1. |
| " | B | " | 1' - 6½" | west | " | " | " | " | " |
| " | C | " | 2' - 9" | east | " | " | " | " | " |
| " | D | " | 2' - 2" | west | " | " | " | " | " |

### Collision Strut

Section B is 2' - 1½" above working point at L1.

### Diagonal U1L2.

Section A is 3' - 0¾" below working point at U1.
" B " 2' - 7¼" above " " " "
Note-Section B has only gage lines numbered 1',3,6,8'.

### Diagonal U2L3.

Section A is 2' - 8¾" below working point at U2.
Note-Section A has only gage lines numbered 1,2,4,5,7,8.

### Floor Beams.

Sections B are 1' - 9" from working points at L1 and L2.
" A " at center line of roadway.

## Hip Vertical.

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Section | A | is | 2' - 6¼" | below | working | point | at | U1. | |
| " | B | " | 1' - 10" | above | " | " | " | L1. | |

## Vertical U2L2.

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Section | A | is | 2' - 2½" | below | working | point | at | U2. |
| " | B | " | 1' - 10" | above | " | " | " | L2. |

## Knee Brace.

Section A is 1' - 8¼" below working point at Head Strut

## Portal Head Strut.

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Section | A | is | 1' - 10" | south | of | working | line | in | N. | Truss. |
| " | H | " | 9' - 7½" | " | " | " | " | " | " | " |

## Diagonal Portal Strut.

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Section | I | is | 1' - 7½" | below | working | line | in | Head | Strut. |
| " | F | " | 5' - 9¾" | " | " | " | " | " | " |
| " | E | " | 8' - 0¼" | " | " | " | " | " | " |
| " | C | " | 13' - 2¼" | " | " | " | " | " | " |

## Auxillary Portal Braces.

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Section | B | is | 1' - 1" | below | working | point | at | U1. | | |
| " | D | " | 1' - 4" | above | " | " | in | center | of | Diag. |
| " | G | " | 1' - 3" | south of | " | " | " | " | " | Strut. |

## Head Strut U2U2!

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Section | A | is | 2' - 7¾" | north | of | working | line | in | S. | Truss. |
| " | B | " | 4' - 7½" | " | " | " | " | " | " | " |
| " | C | " | 5' - 7½" | " | " | " | " | " | " | " |
| " | D | " | at center line of bridge. | | | | | | | |
| " | E | " | 5' - 8½" | south | of | working | line | in | N. | Truss. |
| " | F | " | 4' - 6¼" | " | " | " | " | " | " | " |
| " | G | " | 1' - 12" | " | " | " | " | " | " | " |

Note.- Gage lines on Truss Members are numbered so that the larger numbers are on the outside flanges. On transverse bracing the larger numbers are on the west flanges of the members in every case except that of the Head Strut and Knee Brace at U2U2', in which case the opposite is true.

## RESULTS OF STRAIN GAGE WORK

Primary Stresses. First it may be well to define the term primary stresses. By primary stresses we mean, those direct tensile or compressive stresses in the members, produced by simple truss action. No stresses produced by the bending of truss members, will be considered as primary stresses. This same will apply to all of the other members of the bridge, altho this statement does not hold in the case of the floor beams. Most of the stress in the floor beams is direct flexural stress but this part of the bridge will be discussed later under the heading of secondary stresses.

Table I is a comparison of the observed and computed primary stresses in the truss members. These are the stresses in the members of the truss on which gage readings were taken and are for a panel load of 17,000 pounds, due to the weight of the concrete floor. The computed stresses were computed in the ordinary manner and the observed stresses are the averages of the stresses observed about each section. The curves in Figs. 7 to 10 are the stresses observed at the various gage lines, with readings taken at sections spaced at 8-inch intervals longitudinally along the end post and upper chord members. The curves are drawn to represent average values, and in some cases where the points behave in an erratic manner, the curves are drawn to correspond with those drawn for similar positions on opposite flanges of the member.

# TABLE I.

## COMPARISON OF OBSERVED AND COMPUTED PRIMARY STRESSES DUE TO WEIGHT OF CONCRETE FLOOR

| Member | Total Stress | Section | Actual Area | Unit Stresses Computed | Unit Stresses Observed |
|---|---|---|---|---|---|
| L0U1 | -61500 | 2 [s 10" @ 20# 1 Pl. 15" X 5/16" | 16.40 | -3750 | -3290 |
| U1U2 | -70000 | 2 [s 10" @ 15.3# 1 Pl. 15" X 5/16" | 13.62 | -5130 | -3960 |
| U2U1 | -70000 | 2 [s 10" @ 15.3# 1 Pl. 15" X 5/16" | 13.62 | -5130 | -5850 |
| U2U3 | -84000 | 2 [s 10" @ 15.3# 1 Pl. 15" X 7/16" | 15.50 | -5420 | -6430 |
| L0L1 | 42000 | 2 Ls 6" X 3½" X 3/8" | 6.84 | 6140 | 5710 |
| L1L2 | 42000 | 2 Ls 6" X 3½" X 3/8" | 6.84 | 6140 | 4820 |
| L2L3 | 70000 | 2 Ls 6" X 4" X 9/16" | 10.62 | 6590 | 3620 |
| U1L1 | 17150 | 2 [s 8" @ 11.5# | 6.72 | 2550 | ---- |
| U2L2 | -17150 | 2 [s 8" @ 11.5# | 6.72 | -2550 | ---- |
| U1L2 | 41000 | 2 Ls 6" X 3½" X 7/16" | 7.94 | 5170 | 4290 |
| U2L3 | 20500 | 2 Ls 4" X 3" X 5/16" | 4.18 | 4900 | 5070 |

It is a recognized fact that individual strain gage readings lack precision and therefore any conclusions arrived at must be based on the whole and not on any particular set of readings. There are only a few places where the curves are drawn with a seeming disregard for the observed stresses. This is a condition which can only be remedied as suggested. Strain gages are conceded a precision of from 500 to a 1000 pounds per square inch, and altho the errors in the readings are generally compensating, this is not always the case and the interpretation of any particular group of readings must be based on all the available data at hand.

Figs. 11 to 14 are curves showing the distribution of stress, about the members at the various sections. The points are the actual readings while the curves are drawn with the curves in Figs. 7 to 10 as a basis. The average values of the curves and points are very nearly the same and are given in Table I. No distribution curves were drawn for other members of the truss, since in the time allotted to the work, it was impossible to make a complete study of all of the members.

The results obtained in this part of the investigation are very satisfactory for the most part. The observed and computed stresses in the end post, diagonals and lower chord member L0L1, check within the limits of precision. In the lower chord members L1L2 and L2L3 the observed stresses are considerably lower than the computed, as the stringers nearer the center of the span become more effective in reducing the stresses in these members. This has the opposite effect on the upper chord members and the observed stresses in U2U3 are slightly higher than the computed stresses. This condition can be accounted for in the following manner. If the lower chords and floor members be considered as acting as a unit with the floor system taking its allotted share of the lower chord stresses, the gravity axis of the unit will be higher than it would be if the lower chords were considered as taking all of the stress. This would reduce the effective depth of the truss and the stresses in the upper chords would necessarily be greater.

Readings were taken on the verticals U1L1 and U2L2, but since the stress in these members is small and produced partly by bending in transverse planes, it was impossible to obtain the stress in these members with the few gage lines which were located on them. For this reason no further mention will be made of the action of these members.

A peculiar condition which is hard to account for, exists in the upper chord member U1U2. Stress measurements indicate that the primary stresses are approximately 2000 pounds higher at the U2 end of this member than at the U1 end. The averages of the stresses observed at the lower flanges and center line of the channels, vary by about 1000 pounds per square inch, while those in the cover plate vary by about 3000 pounds per square inch. The distribution curves for the U2 end of this member indicate that there is a considerable amount of bending in the member, since the stresses in the different flanges vary by about 4000 to 5000 pounds per square inch. This together with the possible lack of precision in readings on the cover plate are offered as possible reasons for this inconsistency.

Secondary Stresses. These stresses may be of two kinds, those in the planes of the trusses due to the rigidity of end connections and those in the verticals and transverse members due to the rigid floor beam and lateral connections. This investigation has to do primarily with those first mentioned, altho some readings were taken on the transverse bracing and floor beams at the first two panel points.

The secondary stresses in the planes of the trusses were obtained from the distribution curves in Figs. 11 to 14. The points shown as open circles in Figs. 15 to 18, are the differences between individual readings at each gage line at a section, and the primary stresses at this section, while those shown as closed circles are the differences between the individual readings and the average primary stresses observed at the three sections at the end of each member. In most cases these curves show that the secondary stresses increase at the sections nearer the working points. Even though the stresses are small the curves show a decided tendency in this direction and this could be proved more conclusively by making a more complete study of the action of the members.

Table II is a comparison of the observed and the computed secondary stresses in the upper chords and end post. These

## TABLE II.

### COMPARISON OF OBSERVED AND COMPUTED SECONDARY STRESSES IN UPPER CHORDS AND END POST

| Member | | Computed Secondary Stress at Joints | Computed Secondary Stress at Gage Lines | Observed Secondary Stress |
|---|---|---|---|---|
| End Post L0U1, at U1 | T | +748 | +366 | +280 |
| | B | -1276 | -624 | ----- |
| Upper Chord U1U2, at U1 | T | +853 | +625 | +470 |
| | B | -1454 | -1070 | -300 |
| Upper Chord U2U1, at U2 | T | -657 | -477 | -1070 |
| | B | +1121 | +814 | +1610 |
| Upper Chord U2U3, at U2 | T | -307 | -191 | +670 |
| | B | +507 | +315 | -1100 |

Note- T and B are the upper and lower fibers of the members.

## TABLE III.

### COMPARISON OF OBSERVED AND COMPUTED FLOOR LOAD STRESSES IN TRANSVERSE BENTS

| Member | Computed Stress | | Observed |
|---|---|---|---|
| | 1st. Method | 2nd. Method | Stress |
| Transverse Bent at U2L2 | | | |
| Head Strut Lower Angles | -487 | -562 | -2447 |
| Upper Angles | +401 | +466 | +1500 |
| Floor Beam Lower Flanges | +6060 | +6120 | +4713 |
| Upper Flanges | -6040 | -6100 | ----- |
| Transverse Bent at U1L1 | | | |
| Head Strut Lower Angles | -775 | | +120 |
| Upper Angles | +705 | | +2300 |
| Floor Beam Lower Flanges | +6220 | | +4170 |
| Upper Flanges | -6200 | | ----- |
| Portal Bent at U1L0 | | | |
| Head Strut Lower Angles | -775 | | +120 |
| Upper Angles | +705 | | +2300 |
| End Post at Portal Conn. | -3750 | | -3290 |
| Diag. Strut Lower Sect. | 0 | | +420 |
| Upper Sect. | 0 | | +2040 |
| Aux. Struts Diagonal | 0 | | -2290 |
| Horizontal | 0 | | -2290 |

check as nearly as could be expected in every case, except that of the upper chord member U2U3, in which the observed secondary stresses are opposite in sign from the computed. This might be caused by inaccurate shop work or by lack of precision in stress measurements but whatever the cause the stresses are too low to cause much uneasiness. Fig. 19 shows the type of bending in the members as drawn from the observed secondary stresses. The upper view in each case shows the bending in the plane of the truss while the lower view shows the bending in the plane of the upper chords.

Table III is a comparison of the observed and computed floor load stresses in the members of the transverse bents and portal. The stresses given are the stresses measured and computed at the center lines of the members. Figs. 20 to 22 are curves showing the observed stresses along the head strut at the second panel point. Fig. 20 shows the average stresses in the upper and lower angles of this member. The knee braces of this bent are connected between the sections B & C and E & F. Had it been possible to take readings on the knee braces, it would have been interesting to note the amount of stress necessary to produce the amount of bending observed at B & C. The other knee brace does not seem to have had a great effect on the head strut and would likely have shown very little stress. This bending at sections B & C is carried on thru the head strut and results in higher observed stresses at the center line of the member. This accounts for the differences in observed and computed stresses in this member as given in Table III. The head strut of the transverse bent at the first panel point tends to pull the point U1 inward. This tendency is shown in Fig. 19 and by the observed stresses in Table III and accounts for the differences in the observed and computed stresses in this member. The observed and computed stresses in the floor beams do not check very closely, but this is to be expected, since in setting the concrete may tend to relieve the beams of part of their load. The stresses in the portal members are chiefly bending and are low so that they cause very little concern.


FIG. 5. VIEWS SHOWING MANNER IN WHICH WIND LOAD WAS APPLIED.

## INVESTIGATION OF WIND STRESSES IN PORTAL.

Object of Investigation. The object of this part of the investigation was to study the action of the portal members when loaded to represent the action of the wind. This included a complete study of the stress distribution in the end post and a partial study of the stress distribution in the remaining portal members.

Instrument. The instrument used for this work was the McCollum-Peters Electric Telemeter, devedoped by the U. S. Bureau of Standards, and the property of the American Society of Civil Engineers. We are indebted to the Special Committee on Impact in Highway Bridges of the American Society of Civil Engineers, for the use of this instrument. A complete description of this instrument is given in "United States Bureau of Standards Bulletin 247."

Load. The load used to represent a wind load was applied horizontally at the panel point Ul and was about 8,000 pounds. This is approximately the wind load for which the portal members were designed. As shown in the small view in Fig. 5, the contour of the ground near the east end of the bridge was such as to make this investigation possible. The tree marked T in this photo is located at about the elevation of the panel point Ul and is almost exactly in line with the portal head strut. This afforded an excellent anchorage for the hoist and cable used in applying the load as is shown in the large view

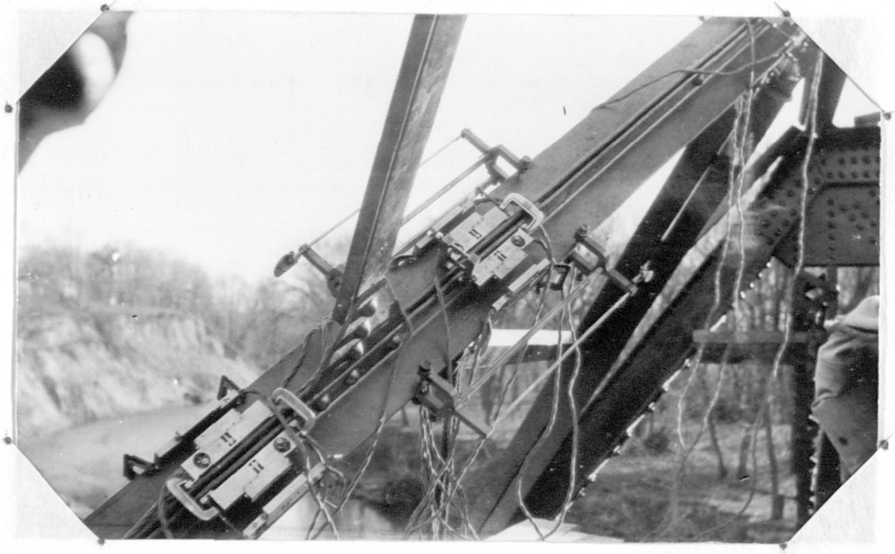


FIG.6. VIEWS SHOWING MANNER IN WHICH B.S. GAGES WERE ATTACHED TO PORTAL MEMBERS.

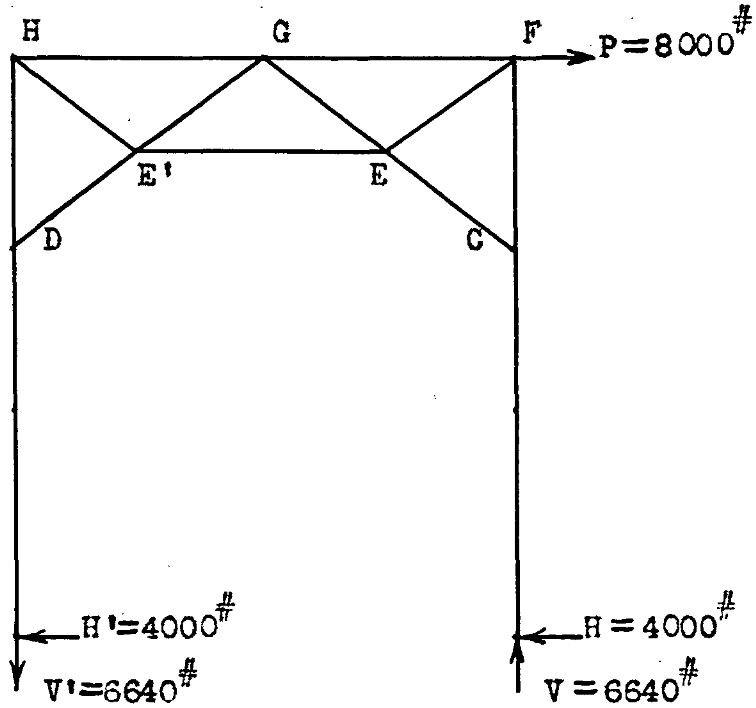
of Fig. 5. A $\frac{3}{4}$-inch bar was placed between the hoist and the cable for the purpose of measuring the applied load. This photo shows the arrangement and the manner in which 2o-inch West extensometers were used for measuring the load. After the field work was completed this bar was taken into the laboratory and calibrated in a testing machine. Fig.1 shows the manner in which the cable was fastened at Ul.

Stress Measurements. Figs. 23 and 24 show the observed stresses in the outer and inner upper flanges of the end post. Readings were also taken on the lower flanges but since the portal strut is connected only to the upper flange, the stress is not distributed uniformly in this member and the observed stresses in the lower flanges are only about 50 per cent of the observed stresses in the upper flanges. The stresses were measured at points 12 inches apart all along the end post, and the curves show a marked similiarity. The points designated as open circles are for a 4000 pound load while those shown as closed circles are for the 8000 pound load. The points of inflection are found at approximately the same place in both flanges and very little difference in stress is noticeable at corresponding points on the flanges. In making these measurements the gages were attached to the members as shown in Fig. 6.

Table IV is a comparison of the observed and computed stresses in portal members due to this horizontal load. These do not check very closely but this is to be expected since

## TABLE IV.

### COMPARISON OF OBSERVED AND COMPUTED WIND LOAD STRESSES IN PORTAL MEMBERS




| Member | Wind Load Stresses | |
|---|---|---|
| | Computed | Observed |
| End Post at C | 7320 | 3660 |
| Portal Strut at C | -2400 | -480 |
| below E | -2400 | -840 |
| above E | -2400 | -830 |
| at G | -2400 | -1830 |
| Head Strut at F | +2710 | +1510 |
| at G | +2710 | +2024 |
| Aux. Strut E-F at F | 0 | -310 |
| at E | 0 | -580 |
| Aux. Strut E-E' at E | 0 | +270 |

part of the load is undoubtedly taken by the lateral bracing and transfered to the transverse sway bracing to be carried down to the plane of the lower chords. Readings taken with the 8-inch West gage checked the Electric Telemeter measurements very closely.

## TABLE V.

PROPERTIES OF TRUSS MEMBERS

| Member | Area Sq.In. | Section | Length In. | I $In.^4$ | C In. |
|---|---|---|---|---|---|
| 1-2 | 6.84 | 2 ∠s 6″ X $3\frac{1}{2}$″ X 3/8″ | 205.714 | 26.10 | 2.25<br>3.75 |
| 2-4 | 6.84 | 2 ∠s 6″ X $3\frac{1}{2}$″ X 3/8″ | 205.714 | 26.10 | 2.25<br>3.75 |
| 4-6 | 10.62 | 2 ∠s 6″ X 4″ X 9/16″ | 205.714 | 39.22 | 2.25<br>3.75 |
| 6-8 | 13.68 | 4 ∠s 6″ X $3\frac{1}{2}$″ X 3/8″ | 205.714 | 52.20 | 2.25<br>3.75 |
| 1-A | 16.40 | 2 [s 10″ @ 20#<br>1 Pl. 15″ X 5/16″ | 162.652 | 246.08 | 3.81<br>6.50 |
| A-3 | 16.40 | 2 [s 10″ @ 20#<br>1 Pl. 15″ X 5/16″ | 162.652 | 246.08 | 3.81<br>6.50 |
| 3-5 | 13.62 | 2 [s 10″ @ 15.3#<br>1 Pl. 15″ X 5/16″ | 205.714 | 216.63 | 3.81<br>6.50 |
| 5-7 | 15.50 | 2 [s 10″ @ 15.33#<br>1 Pl. 15″ X 7/16″ | 205.714 | 244.78 | 3.94<br>6.50 |
| 7-9 | 15.50 | 2 [s 10″ @ 15.3#<br>1 Pl. 15″ X 7/16″ | 205.714 | 244.78 | 3.94<br>6.50 |
| 3-4 | 7.94 | 2 ∠s 6″ X $3\frac{1}{2}$″ X 7/16″ | 325.304 | 29.88 | 2.25<br>3.75 |
| 5-6 | 4.18 | 2 ∠s 4″ X 3″ X 5/16″ | 325.304 | 9.08 | 2.00 |
| 7-8 | 2.38 | 2 ∠s $2\frac{1}{2}$″ X $2\frac{1}{2}$″ X 1/4″ | 325.304 | 2.42 | 1.38<br>1.12 |
| A-2 | 2.88 | 2 ∠s $3\frac{1}{2}$″ X $2\frac{1}{2}$″ X 1/4″ | 162.652 | 4.78 | 1.75 |
| Posts | 6.72 | 2 ∠s 8″ @ 11.5# | 252.000 | 66.34 | 3.08 |

## COMPUTATION OF SECONDARY STRESSES

Secondary Stresses in Truss. The method used in computing the secondary stresses in the truss, is a simplification of Manderla's method, as given in Johnson, Bryan and Turneaure's, "Framed Structures" Part II. No attempt will be made at giving a complete review of this method and this discussion will be confined chiefly to results.

Table V gives the properties of the truss members, necessary in calculating the secondary stresses, and Fig. 25 the floor load stresses in the truss. (A) in Fig. 25 is the truss with a collision strut, while in (B) the collision strut has been left out. It was thot that a study of the effect of the collision strut on secondary stresses would be beneficial and so these stresses were computed for both conditions.

Tables VI and VII give the equations as formulated for the different joints. Table VI is for the truss with a collision strut while Table VII is for it without this member. The solving of these equations simultaneously proved to be a laborious process. A Monroe calculator was used for this work and the figures were carried out to eight places. This may be a greater precision than is necessary altho on the first time thru these equations, the figures were carried two places beyond the decimal point and it was found that it was almost impossible to get the deflection angles, figured from three seperate equations, to check. The calculation of secondary

# TABLE VI.

## EQUATIONS FOR TRUSS WITH COLLISION STRUT

| No. of Joint | First Members of Equations | Absolute Terms |
|---|---|---|
| 1 | $3.279602T_1 + 1.512926T_A + 0.126875T_2$ | $-35121.908$ |
| A | $1.512926T_1 + 6.110444T_A + 0.039370T_2 + 1.512926T_3$ | $-53788.630$ |
| 2 | $0.126875T_1 + 0.029370T_A + 1.092748T_2 + 0.263254T_3 + 0.126875T_4$ | $+14728.086$ |
| 3 | $1.512926T_A + 0.263254T_2 + 5.842190T_3 + 0.091853T_4 + 1.053062T_5$ | $+34779.809$ |
| 4 | $0.126875T_2 + 0.091853T_3 + 1.345076T_4 + 0.263254T_5 + 0.190556T_6$ | $+11965.224$ |
| 5 | $1.053062T_3 + 0.263254T_4 + 5.068262T_5 + 0.027912T_6 + 1.189903T_7$ | $-44943.482$ |
| 6 | $0.190556T_4 + 0.027912T_5 + 1.470944T_6 + 0.263254T_7 + 0.253750T_8$ | $+20786.582$ |
| 7 | $1.189903T_5 + 0.263254T_6 + 5.300998T_7 + 0.007439T_8 + 1.189903T_9$ | $-66717.198$ |
| 8 | $0.253750T_6 + 0.007439T_7 + 1.485822T_8 + 0.263254T_9 + 0.190556T_{10} + 0.02791T_{11}$ | $+12812.985$ |
| 9 | $1.189903T_7 + 0.263254T_8 + 5.286120T_9 + 1.189903T_{11}$ | $-67055.413$ |
| 10 | $0.190556T_8 + 1.345076T_{10} + 0.263254T_{11} + 0.091853T_{13} + 0.126875T_{12}$ | $-3202.823$ |
| 11 | $0.027912T_8 + 1.189903T_9 + 0.263254T_{10} + 5.068262T_{11} + 1.053062T_{13}$ | $-30460.442$ |
| 13 | $0.091853T_{10} + 1.053062T_{11} + 5.842190T_{13} + 0.263254T_{12} + 1.512926T_L$ | $-7725.907$ |
| 12 | $0.126875T_{10} + 0.263254T_{13} + 1.092748T_{12} + 0.029370T_L + 0.126875T_{14}$ | $+3518.179$ |
| L | $1.512926T_{13} + 0.029370T_{12} + 6.110444T_L + 1.512926T_{14}$ | $-66608.112$ |
| 14 | $0.126875T_{12} + 1.512926T_L + 3.279602T_{14}$ | $+14551.594$ |

## TABLE VII.

EQUATIONS FOR TRUSS WITHOUT COLLISION STRUT

| No. of Joint | First Members of Equations | Absolute Terms |
|---|---|---|
| 1 | $1.766676T_1 + 0.126875T_2 + 0.756463T_3$ | $-1513.419$ |
| 2 | $0.126875T_1 + 1.034008T_2 + 0.263254T_3 + 0.126875T_4$ | $+13009.221$ |
| 3 | $0.756463T_1 + 0.263254T_2 + 4.329264T_3 + 0.091853T_4 + 1.053062T_5$ | $-2307.809$ |
| 4 | $0.126875T_2 + 0.091853T_3 + 1.345076T_4 + 0.263254T_5 + 0.190556T_6$ | $+11965.224$ |
| 5 | $1.053062T_3 + 0.263254T_4 + 5.068262T_5 + 0.027912T_6 + 1.189903T_7$ | $-44943.481$ |
| 6 | $0.190556T_4 + 0.027912T_5 + 1.470944T_6 + 0.263254T_7 + 0.253750T_8$ | $+20786.582$ |
| 7 | $1.189903T_5 + 0.263254T_6 + 5.300998T_7 + 0.007439T_8 + 1.189903T_9$ | $-66717.198$ |
| 8 | $0.253750T_6 + 0.007439T_7 + 1.485822T_8 + 0.263254T_9 + 0.190556T_{10} + 0.02791T_{11}$ | $+12812.985$ |
| 9 | $1.189903T_7 + 0.263254T_8 + 5.286120T_9 + 1.189903T_{11}$ | $-67055.413$ |
| 10 | $0.190556T_8 + 1.345076T_{10} + 0.263254T_{11} + 0.091853T_{13} + 0.126875T_{12}$ | $-3202.823$ |
| 11 | $0.027912T_8 + 1.189903T_9 + 0.263254T_{10} + 5.068262T_{11} + 1.053062T_{13}$ | $-30460.442$ |
| 13 | $0.091853T_{10} + 1.053062T_{11} + 4.329264T_{13} + 0.263254T_{12} + 0.756463T_{14}$ | $-11054.894$ |
| 12 | $0.126875T_{10} + 0.263254T_{13} + 1.034008T_{12} + 0.126875T_{14}$ | $+487.015$ |
| 14 | $0.756463T_{13} + 0.126875T_{12} + 1.766676T_{14}$ | $-9976.309$ |

stresses is generally thot of as a laborious and practically endless task, and without a doubt it is rather tedious, altho less so than might be expected. In a truss of this type it would be possible for a person familiar with the methods, to calculate the secondary stresses in approximately 20 hours. It seems that this time would be rather well spent, especially in the case of larger bridges, since the maximum secondary stresses are high enough to cause some concern.

Table VIII is a comparison of the secondary and primary stresses in the truss, with and without a collision strut. From these figures it would seem that the effect of the collision strut on the stresses in the end post is quite important, and it is doubtful if the benefits derived from its use are enough to make its use advisable. Unfortunately no secondary stress measurements were taken on the end post near the collision strut, but those taken at the other points check the computed as closely as could be expected, so that the secondary stress theory can hardly be questioned. The stresses given in Table VIII are the stresses at the joints neglecting the effect of the gusset plates. These tend to stiffen the members and relieve them to a certain extent.

In Table II is given a comparison of the observed and computed secondary stresses at the gage lines at which the observed stresses were measured. The stresses at these points were computed assuming a straight line variation, which altho not exactly correct is doubtlessly accurate enough for

## TABLE VIII.

### COMPARISON OF COMPUTED PRIMARY AND SECONDARY UNIT STRESSES

| Member | | | Secondary Stresses at Joints | | Primary | Percentages Secondary of Primary | |
|---|---|---|---|---|---|---|---|
| | | | Col. Strut | No Strut | Stresses | Col. Strut | No Strut |
| End Post | 1A | T | +42 | -91 | -3750 | 1.1 | 2.4 |
| | | B | -71 | +156 | " | 1.9 | 4.2 |
| | A1 | T | -1034 | | " | 27.6 | |
| | | B | +1763 | | " | 47.0 | |
| | A3 | T | -1039 | | " | 27.7 | |
| | | B | +1771 | | " | 47.2 | |
| | 3A | T | +748 | +41 | " | 19.9 | 1.1 |
| | | B | -1276 | -70 | " | 34.0 | 1.9 |
| Upper Chords | 35 | T | +853 | +274 | -5130 | 16.6 | 5.3 |
| | | B | -1454 | -467 | " | 28.4 | 9.1 |
| | 53 | T | -657 | -469 | " | 12.8 | 9.1 |
| | | B | +1121 | +799 | " | 21.8 | 15.6 |
| | 57 | T | -307 | -171 | -5420 | 5.7 | 3.2 |
| | | B | +507 | +282 | " | 9.4 | 5.2 |
| | 75 | T | -449 | -532 | " | 8.3 | 9.8 |
| | | B | +741 | +876 | " | 13.7 | 16.2 |
| | 79 | T | -356 | -384 | " | 6.6 | 7.1 |
| | | B | +588 | +632 | " | 10.9 | 11.6 |
| Main Diagonal | 34 | T | +53 | -186 | +5170 | 1.0 | 3.6 |
| | | B | -88 | +310 | " | 1.7 | 6.0 |
| | 43 | T | -107 | +9 | " | 2.1 | 0 |
| | | B | +178 | -14 | " | 3.4 | 0 |
| Hip Vertical | 23 | | 195 | 390 | +2550 | 7.6 | 15.3 |
| | 32 | | 69 | 485 | " | 2.7 | 19.0 |

## TABLE VIII (Cont'd).

### COMPARISON OF COMPUTED PRIMARY AND SECONDARY UNIT STRESSES

| Member | | | Secondary Stresses at Joints | | Primary | Percentages Secondary of Primary | |
|---|---|---|---|---|---|---|---|
| | | | Col.Strut | No Strut | Stresses | Col.Strut | No Strut |
| Lower Chords | 12 | T | -386 | +847 | +6140 | 6.3 | 13.7 |
| | | B | +231 | -508 | " | 3.8 | 8.3 |
| | 21 | T | -432 | -1070 | " | 7.0 | 17.4 |
| | | B | +259 | +640 | " | 4.2 | 10.4 |
| | 24 | T | +116 | +150 | " | 1.9 | 2.4 |
| | | B | -70 | -90 | " | 1.1 | 1.5 |
| | 42 | T | -446 | -473 | " | 7.3 | 7.7 |
| | | B | +267 | +283 | " | 4.3 | 4.6 |
| | 46 | T | +724 | +739 | +6590 | 11.0 | 11.2 |
| | | B | -434 | -443 | " | 6.6 | 6.7 |
| | 64 | T | -1110 | -1110 | " | 16.8 | 16.8 |
| | | B | +660 | +666 | " | 10.0 | 10.1 |
| | 68 | T | -345 | -345 | +6140 | 5.6 | 5.6 |
| | | B | +207 | +207 | " | 3.4 | 3.4 |
| Second Vertical | 45 | | 560 | 496 | -2550 | 22.0 | 19.4 |
| | 54 | | 785 | 687 | " | 30.8 | 26.9 |
| Second Diagonal | 56 | | 318 | 271 | +4900 | 6.5 | 5.5 |
| | 65 | | 82 | 55 | " | 1.7 | 1.1 |
| Third Vertical | 67 | | 295 | 304 | 0 | 0 | 0 |
| | 76 | | 294 | 312 | 0 | 0 | 0 |
| Center Diagonal | 78 | T | -129 | -138 | 0 | 0 | 0 |
| | | B | +158 | +169 | 0 | 0 | 0 |
| Collision Strut | A2 | | 112 | | 0 | 0 | |
| | 2A | | 140 | | 0 | 0 | |

purposes of comparison. Figs. 26 (A) and 26 (B) are the diagrams of computed secondary stresses in the truss with and without collision strut. These figures show the type of bending and the effect of the collision strut, but the magnitude of the stress is not given.

Secondary Stresses in Transverse Bents. In computing the secondary stresses in the members of the transverse bents at the first two panel points, the "Area of Moments", method was used. No knee braces are used in the bent at the first panel point, and those used in the bent at the second panel point were considered merely as a means of making the joints rigid. The stresses in the bent at the second panel point were also computed by the "Method of Work", and in this case it was assumed that no bending could occur between the points at which the knee braces were connected to the bents. The stresses were computed for the head struts and floor beams only, since it was on these members that stress measurements were taken. The results of this investigation are given in Table III, and the discussion is taken up elsewhere in this paper.

## CONCLUSIONS

Instruments. In strain gage work a precision of about 500 pounds per square inch is generally assumed. Errors of 1000 pounds per square inch are entirely possible however, and frequently occur. Measurements taken with the Telemeter are more accurate, since the human element in the work is largely eliminated, and a precision of from 200 to 300 pounds per square inch is assumed for this instrument. This instrument is well adapted for stress measurement work and should be used whereever practicable. Its use is limited mainly to the measurement of live load stresses, and its greatest value is found in the fact that it is particularly well adapted for the measurement of stresses produced by moving loads. The strain gage is the only practicable means of measuring dead load stresses, where the investigation is carried on for a considerable period of time. Its precision depends upon the number of readings taken, and the number taken should be such, as to make it possible to eliminate inaccuracies by a process of elimination.

Primary Stresses. The observed and computed primary stresses shown in Table I, check as closely as could be expected. Most of the inconsistencies can be accounted for in the action of the floor members. Inasmuch as the floor tends to reduce the stresses in the lower chord members and increase the stresses in the upper chord members, by reducing the effective depth of the truss. Taken as a whole the stress distribution is good, and it is only at the points at which the secondary stresses are high that there is much difference in stress through-out the section.

Secondary Stresses. Tables II and III are comparisons of observed and computed secondary stresses. Inaccurate shop work or lack of precision in stress measurements may account for some of the inconsistencies, some of which could hardly be called inconsistencies since the stresses are quite low. For the precise measurement of these stresses the number of gage lines should be such, as to make it possible to eliminate errors by a process of elimination. The greatest observed secondary stress was found at the U2 end of the member U1U2, and was about 30% of the primary stress at this point. No measurements were taken on the end post near the collision strut, and so we have no observed secondary stresses, where theoretically they are greatest. The curves in Figs. 15 to 18 show that the secondary stresses increase at the sections nearer the working points, as theoretically they should.

Wind Stresses in Portal. Table IV is a comparison of the observed and computed wind load stresses in portal members. These stresses can hardly be expected to check, since a part of the load is undoubtedly transfered to the plane of the lower chords, through the transverse and lateral bracing. The curves in Figs. 23 and 24 are remarkable for their consistency. As expected the stresses are concentrated in the upper flanges of the end post, and it was found that the lower flange stresses were only about 50% of the upper flange stresses. This condition is caused by connecting the portal strut to the upper flange of the end post, and can easily be remedied by a different type of connection.

As is usually assumed the point of inflection in the end post due to wind stresses, is found approximately midway between the lower working point and the point at which the portal strut is connected to this member. The location of this point depends upon the fixity of the connections of the end post, and in a bridge of this type the above assumption can hardly be questioned.

Computation of Secondary Stresses. The amount of work involved in the calculation of secondary stresses, is not as great as is generally supposed. In all except very small bridges, the calculation of these stresses is important, as it is another step toward eliminating the "factor of ignorance". In this case the effect of the collision strut on the secondary stresses in the end post, is such as to make its use seem unadvisable. The collision strut is directly responcible for increasing the secondary stresses in the end post from about 5% to nearly 50% of the primary stresses in this member. Altho some of the other members of the bridge show an increase in stress when this member is omitted, none of these members are subject to as many loads as the end post, hence it is important that the stresses in this member be reduced as much as possible.

The observed and computed secondary stresses in the members of the transverse bents are low, and it is doubtful if the computation of these stresses is necessary, since those members are usually much larger than actually required.

Form E-5

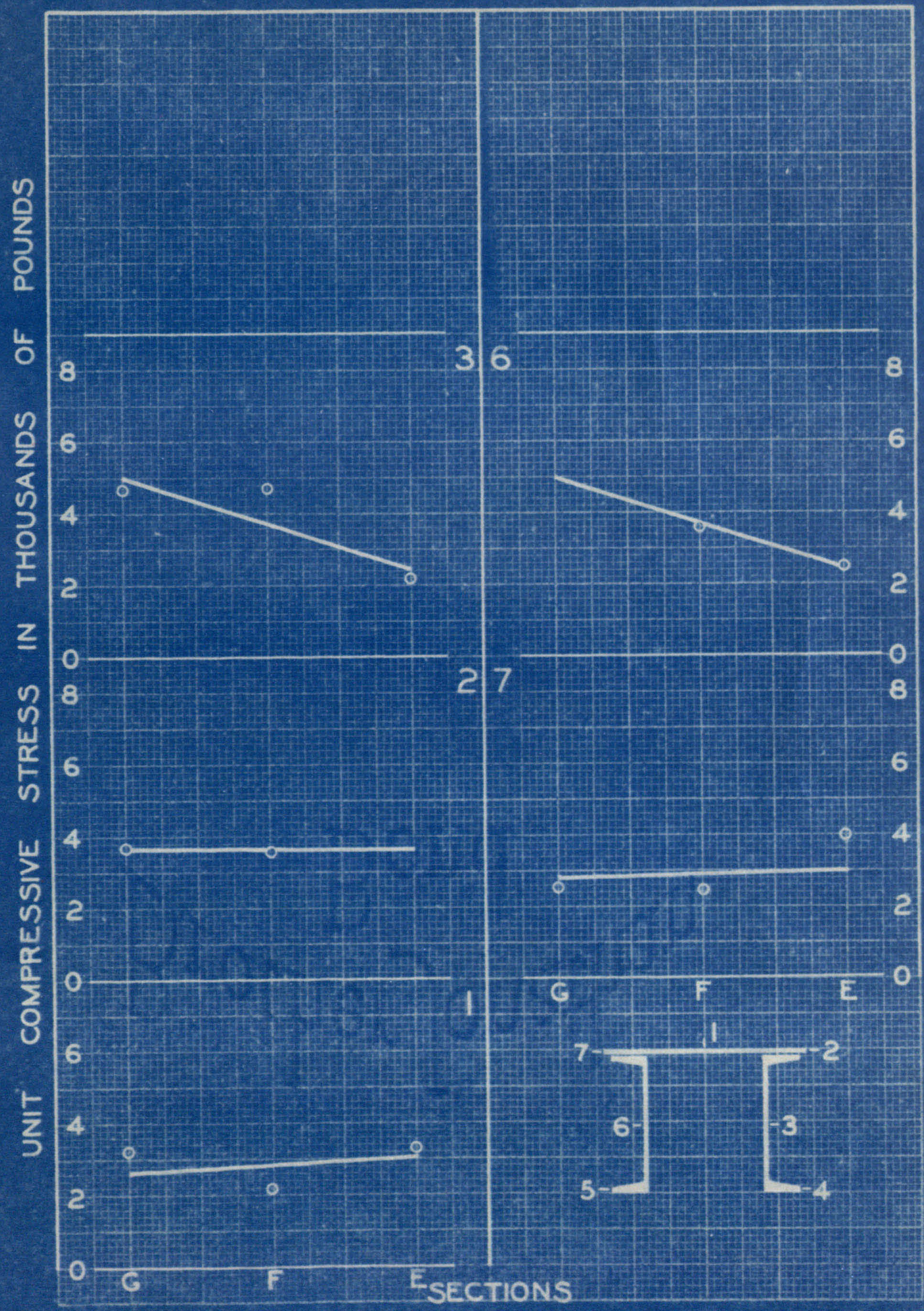



FIG. 7. LONGITUDINAL STRESS DISTRIBUTION IN END POST DUE TO FLOOR LOAD

Form E-5

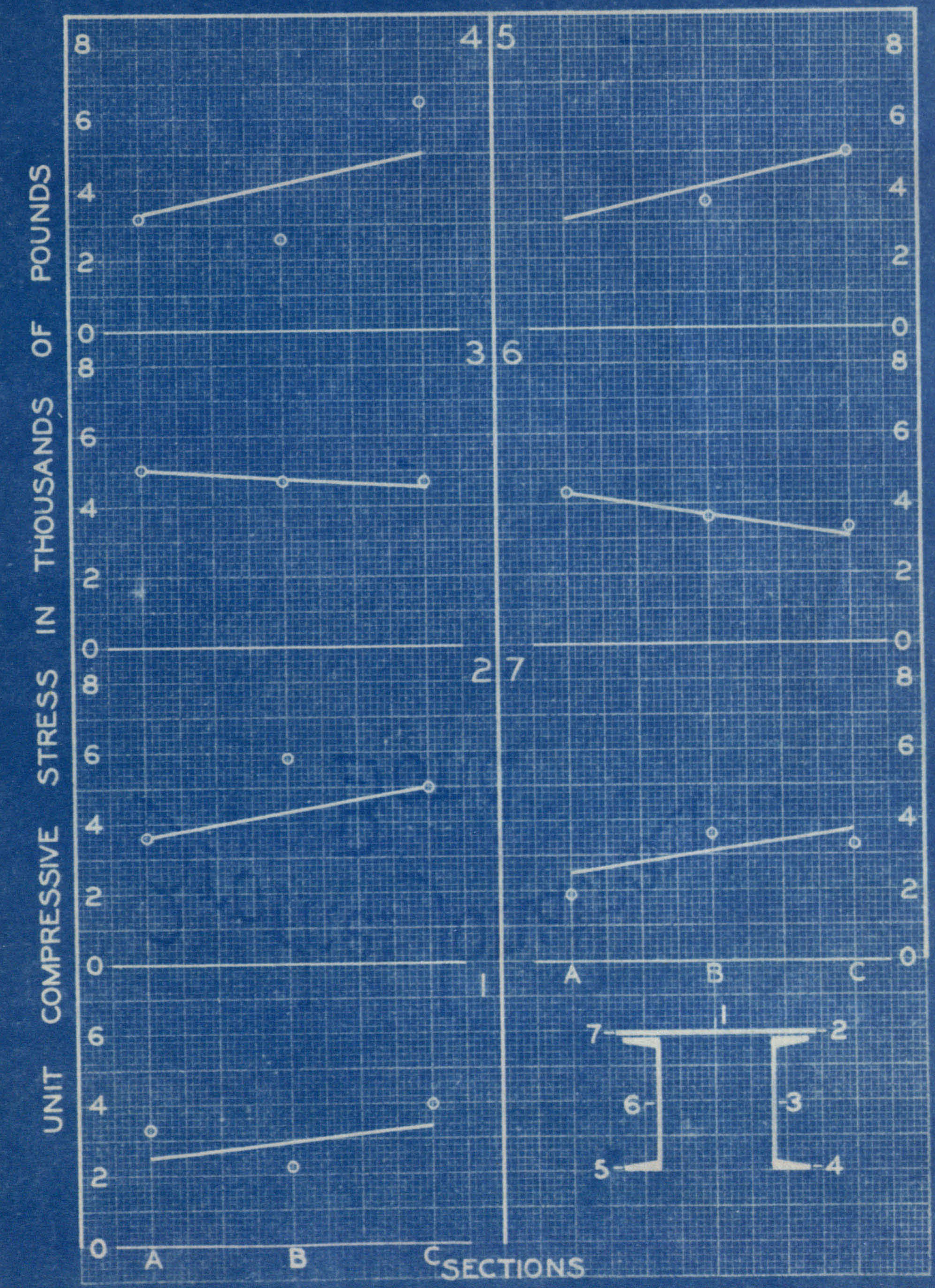



FIG. 8. LONGITUDINAL STRESS DISTRIBUTION IN UPPER CHORD $U_1U_2$ DUE TO FLOOR LOAD

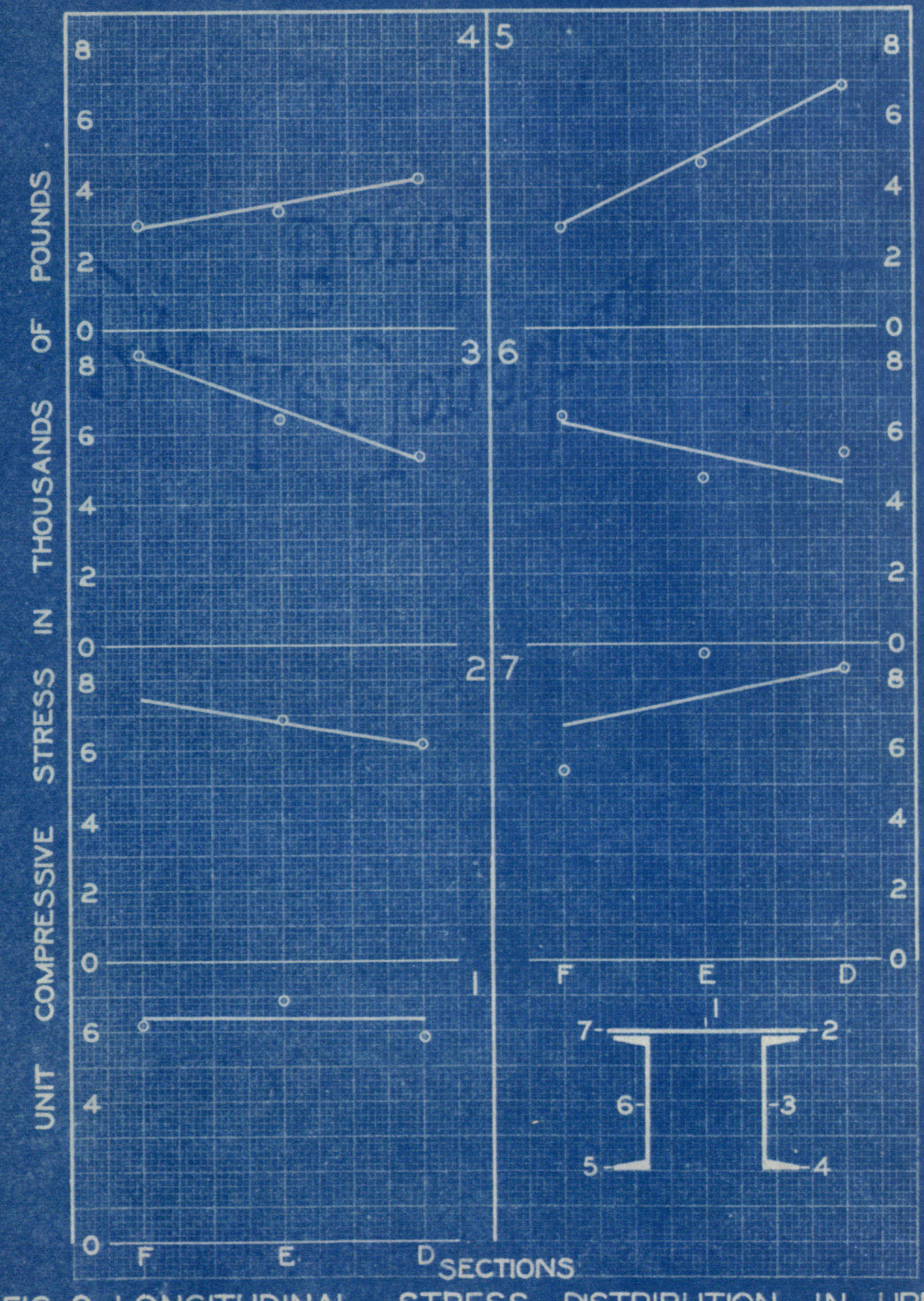



FIG. 9. LONGITUDINAL STRESS DISTRIBUTION IN UPPER CHORD $U_2U_1$ DUE TO FLOOR LOAD

Form E-5

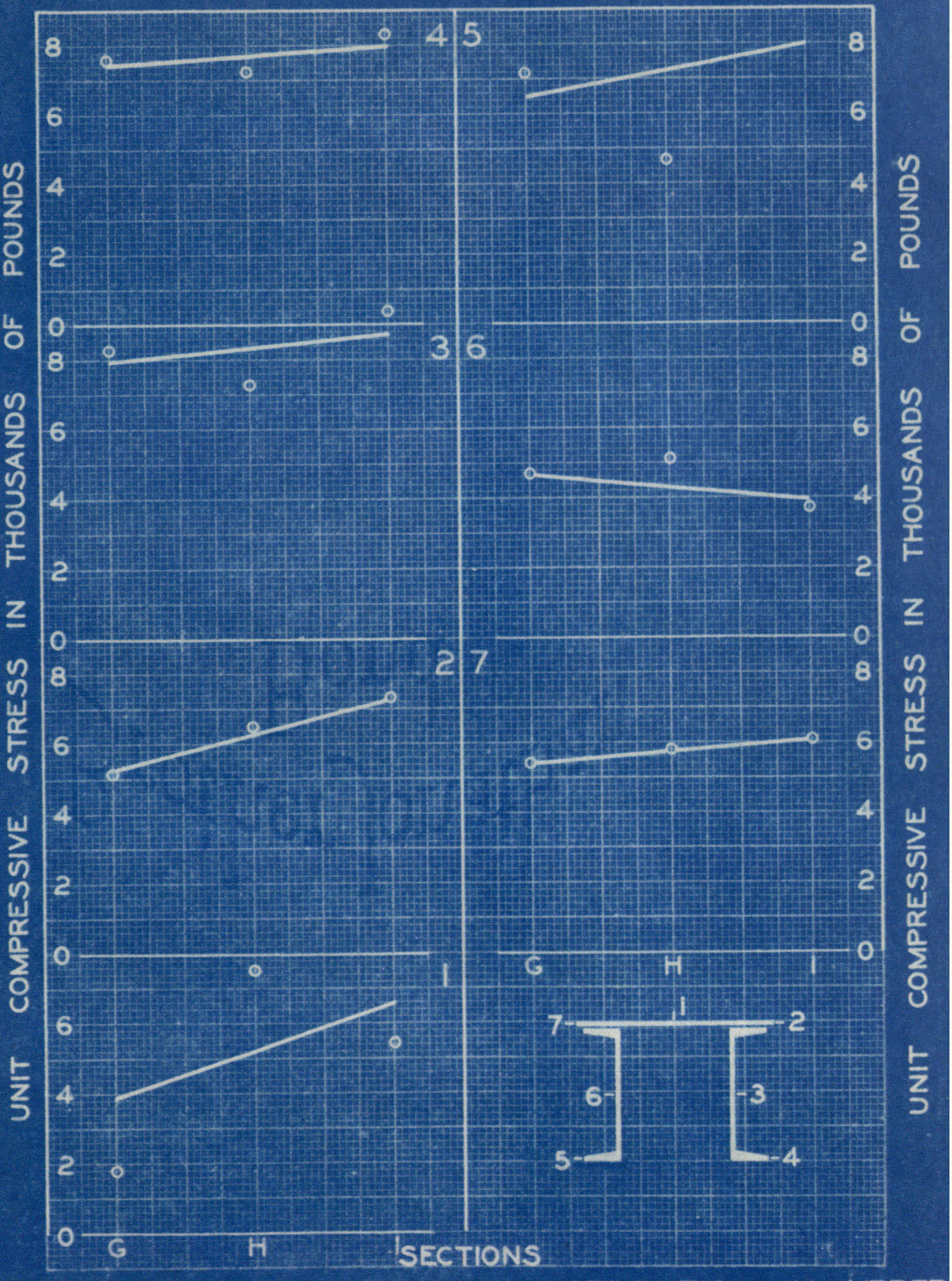



FIG.10. LONGITUDINAL STRESS DISTRIBUTION IN UPPER CHORD $U_2U_3$ DUE TO FLOOR LOAD

Form E-5

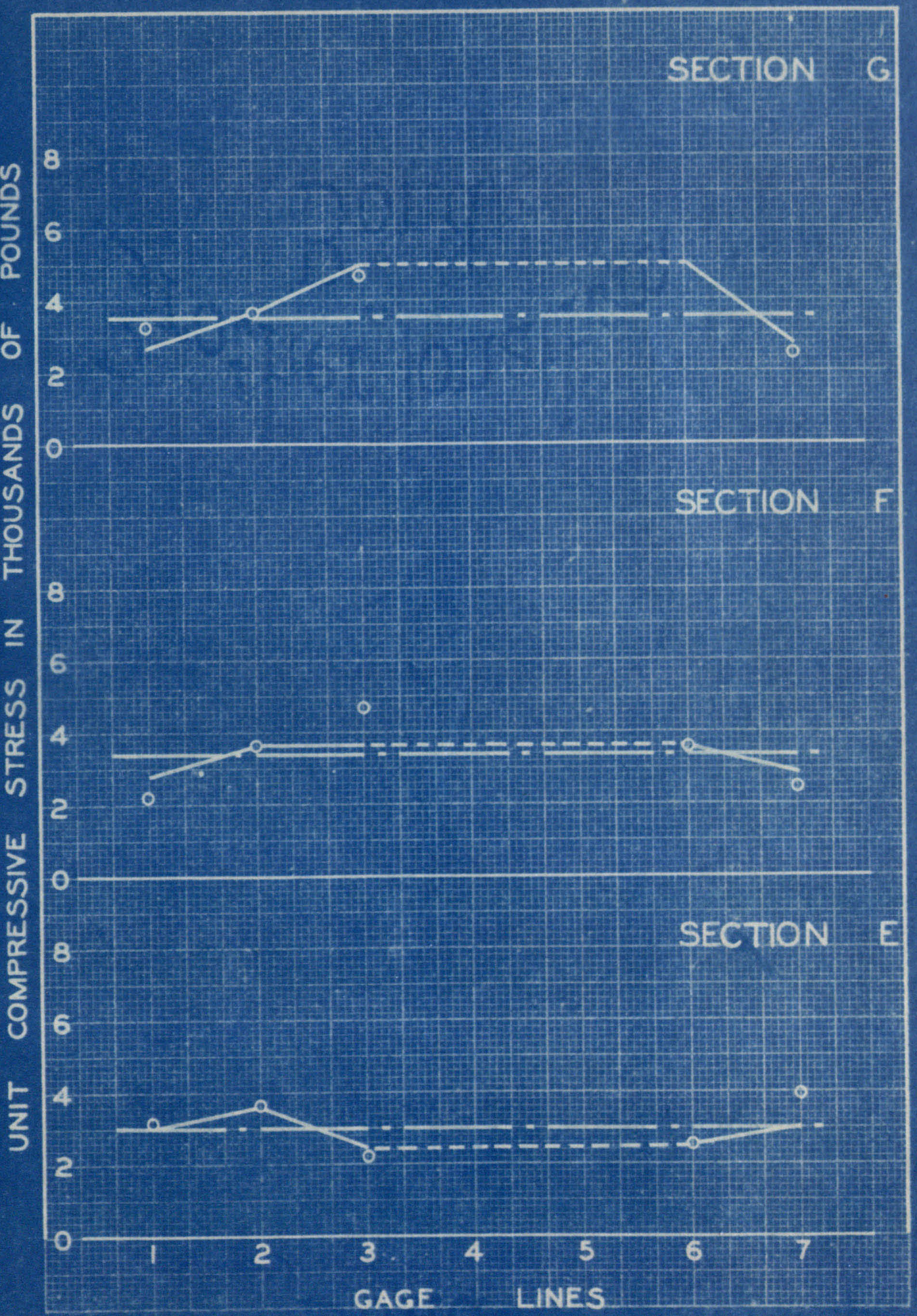



FIG. 11. TRANSVERSE STRESS DISTRIBUTION IN END POST DUE TO FLOOR LOAD

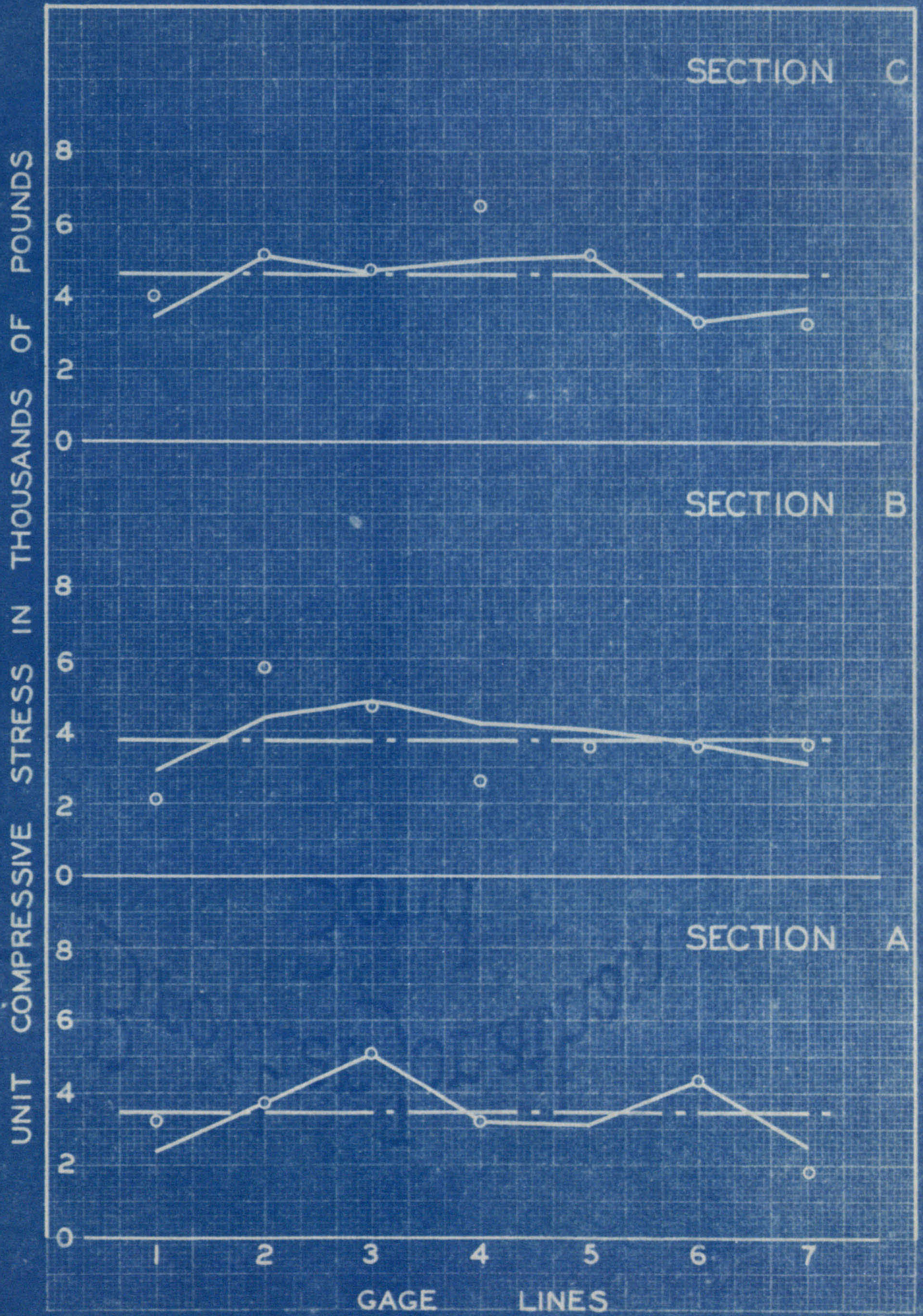



FIG. 12. TRANSVERSE STRESS DISTRIBUTION IN UPPER CHORD $U_1U_2$ DUE TO FLOOR LOAD

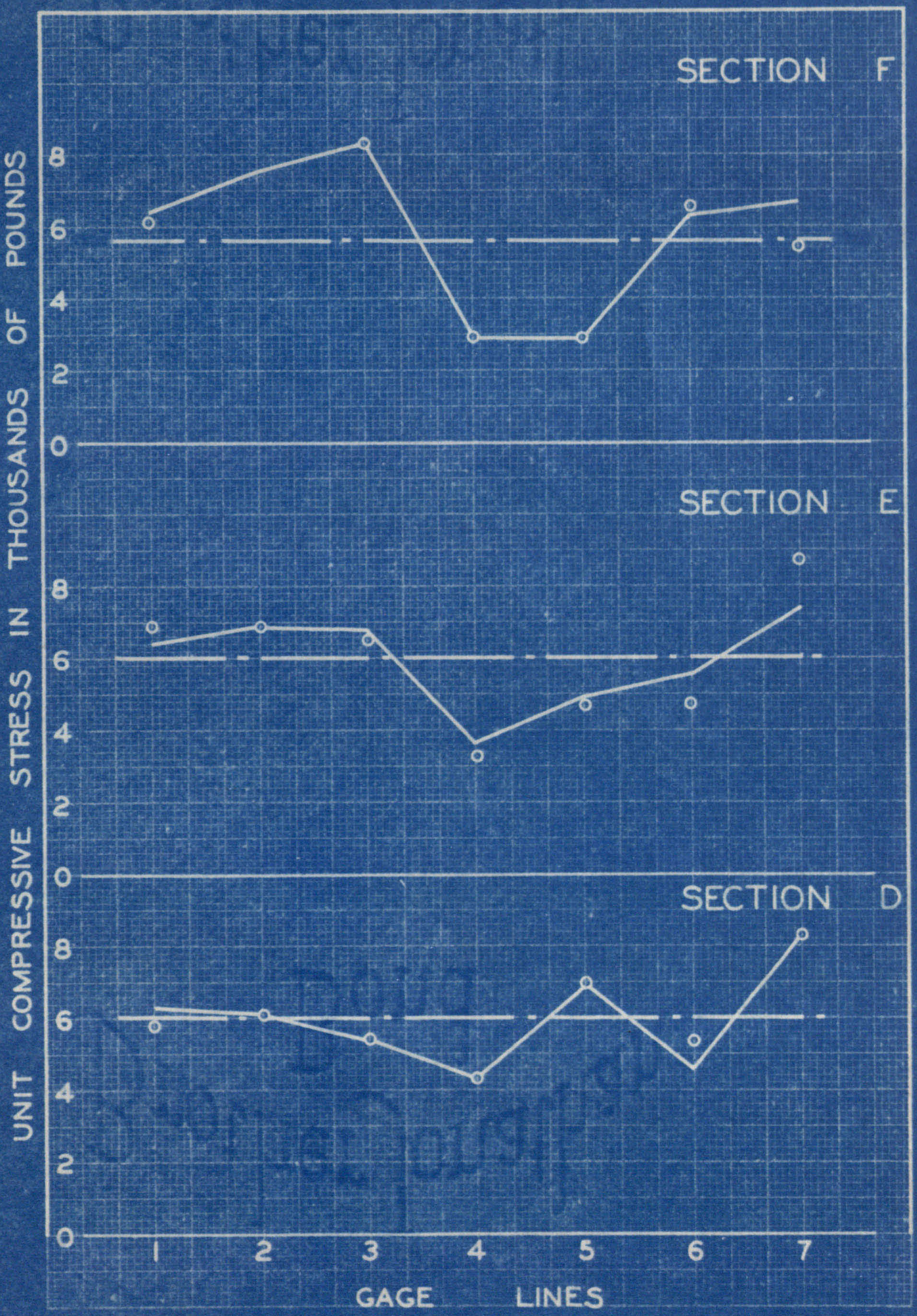



FIG. 13. TRANSVERSE STRESS DISTRIBUTION IN UPPER CHORD $U_2U_1$ DUE TO FLOOR LOAD

Form E-5

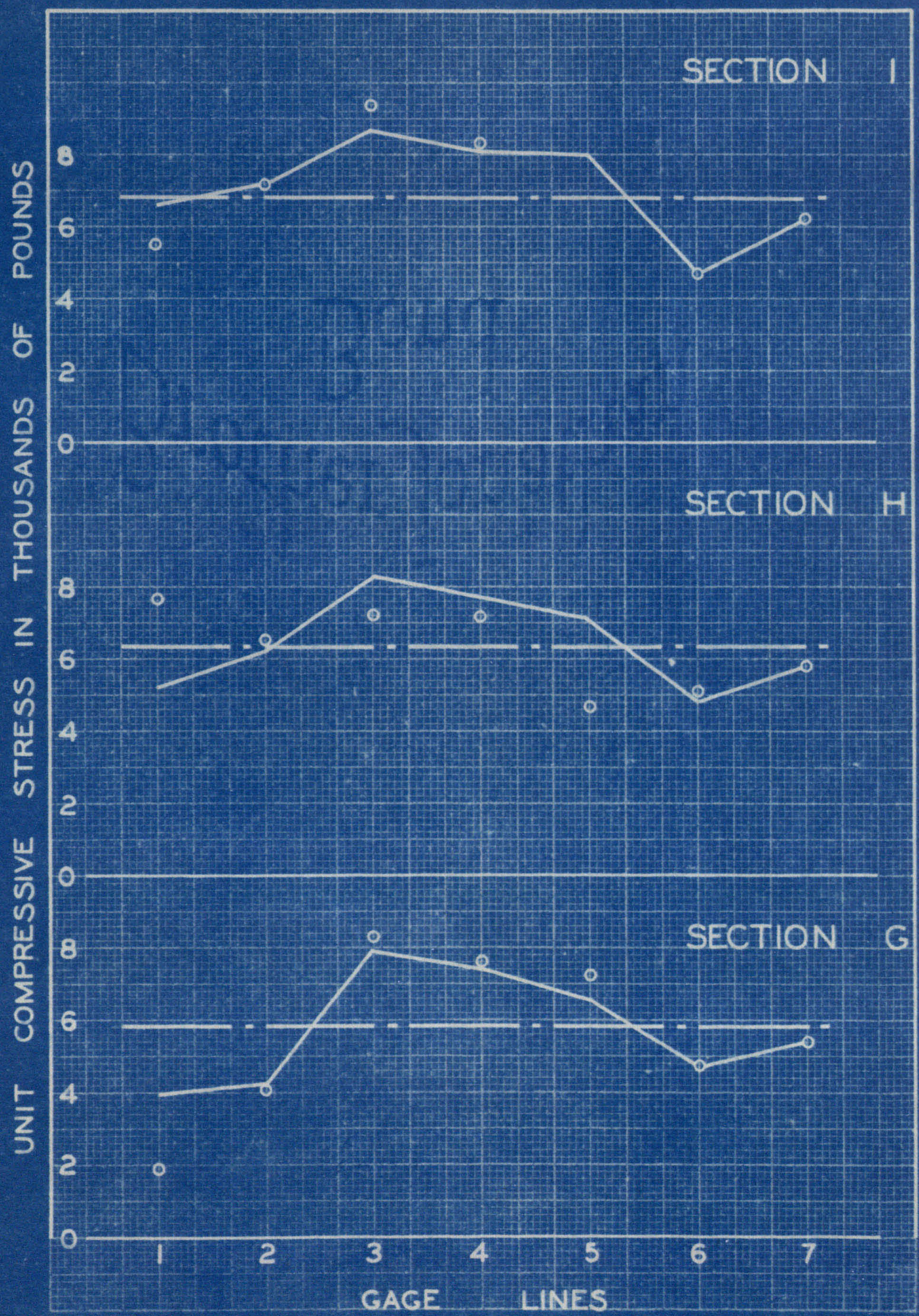



FIG. 14. TRANSVERSE STRESS DISTRIBUTION IN UPPER CHORD $U_2U_3$ DUE TO FLOOR LOAD

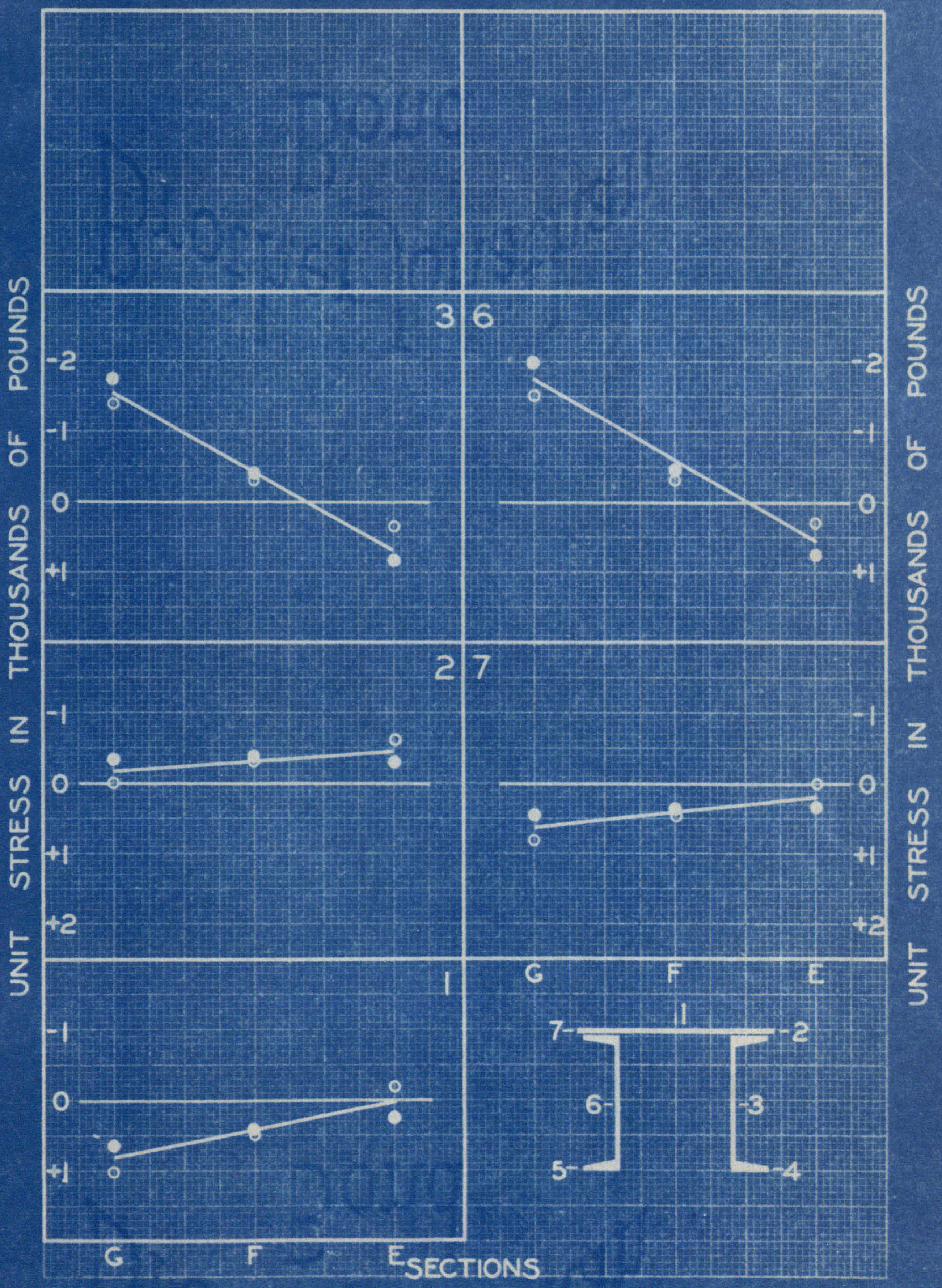



FIG. 15. LONGITUDINAL SECONDARY STRESS DISTRIBUTION IN END POST DUE TO FLOOR LOAD

Form E-5

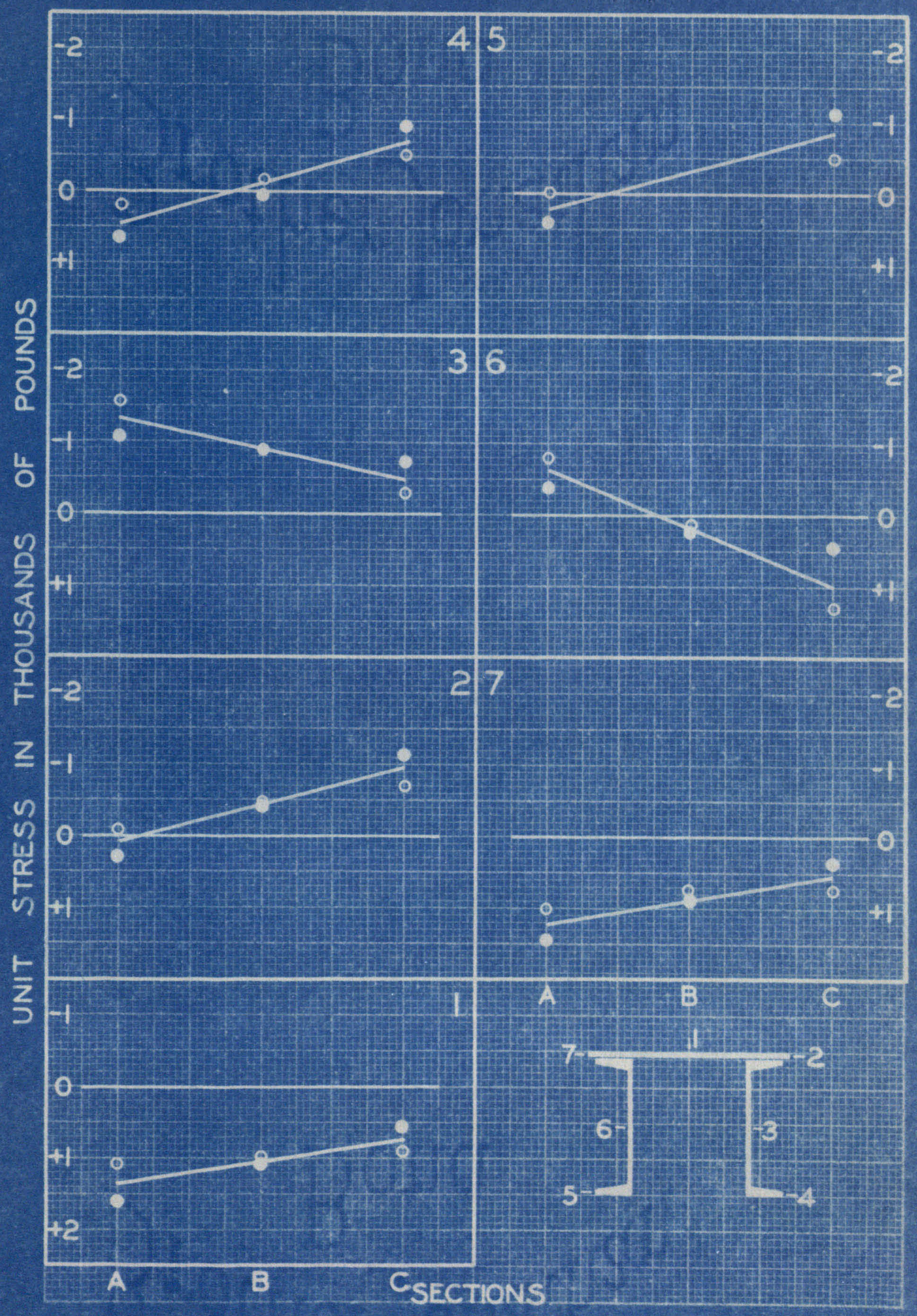



FIG. 16. LONGITUDINAL SECONDARY STRESS DISTRIBUTION IN UPPER CHORD $U_1U_2$ DUE TO FLOOR LOAD

Form E-5

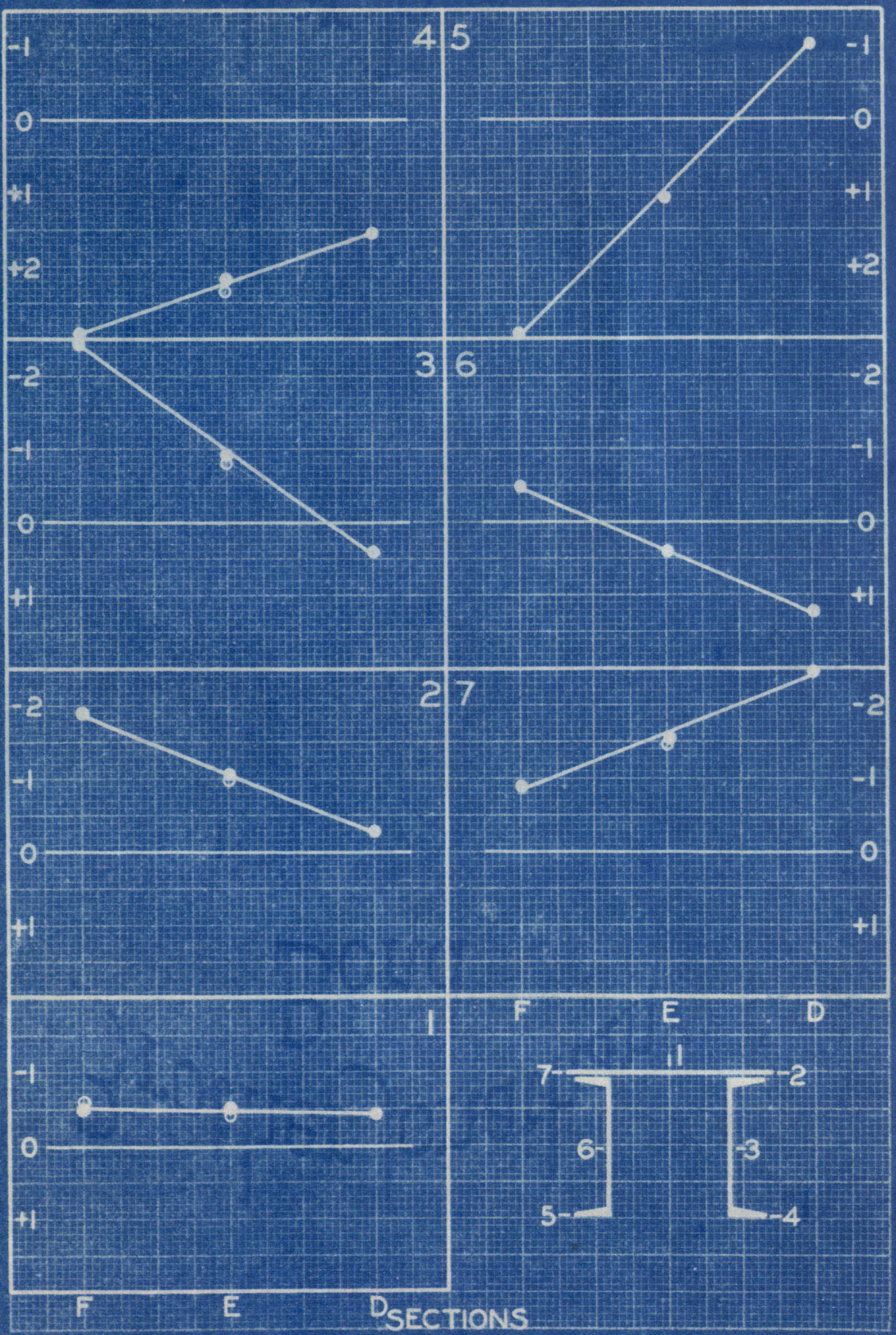



FIG. 17. LONGITUDINAL SECONDARY STRESS DISTRIBUTION IN UPPER CHORD $U_2U_1$ DUE TO FLOOR LOAD

Form E-5

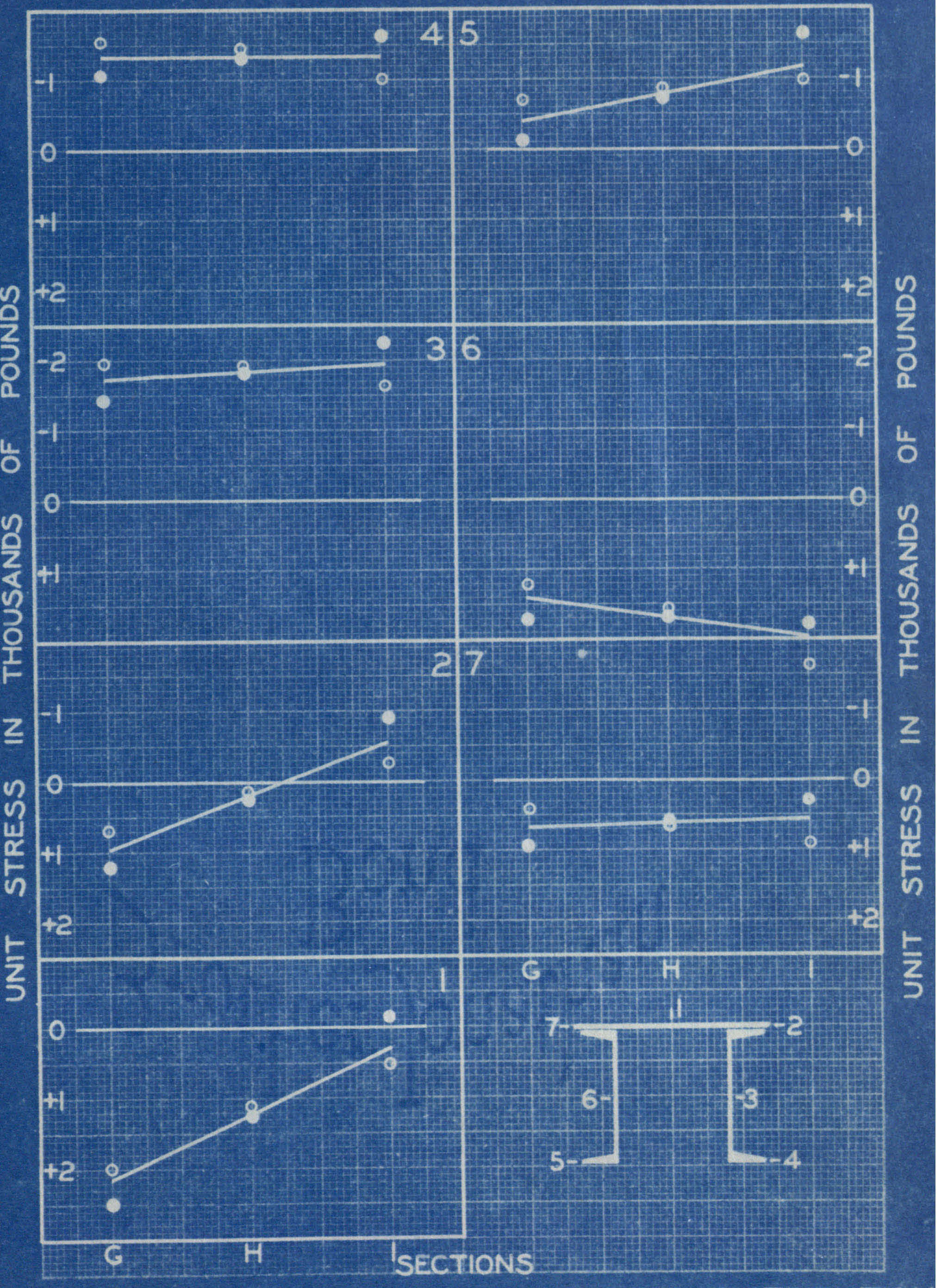



FIG. 18. LONGITUDINAL SECONDARY STRESS DISTRIBUTION IN UPPER CHORD $U_2U_3$ DUE TO FLOOR LOAD

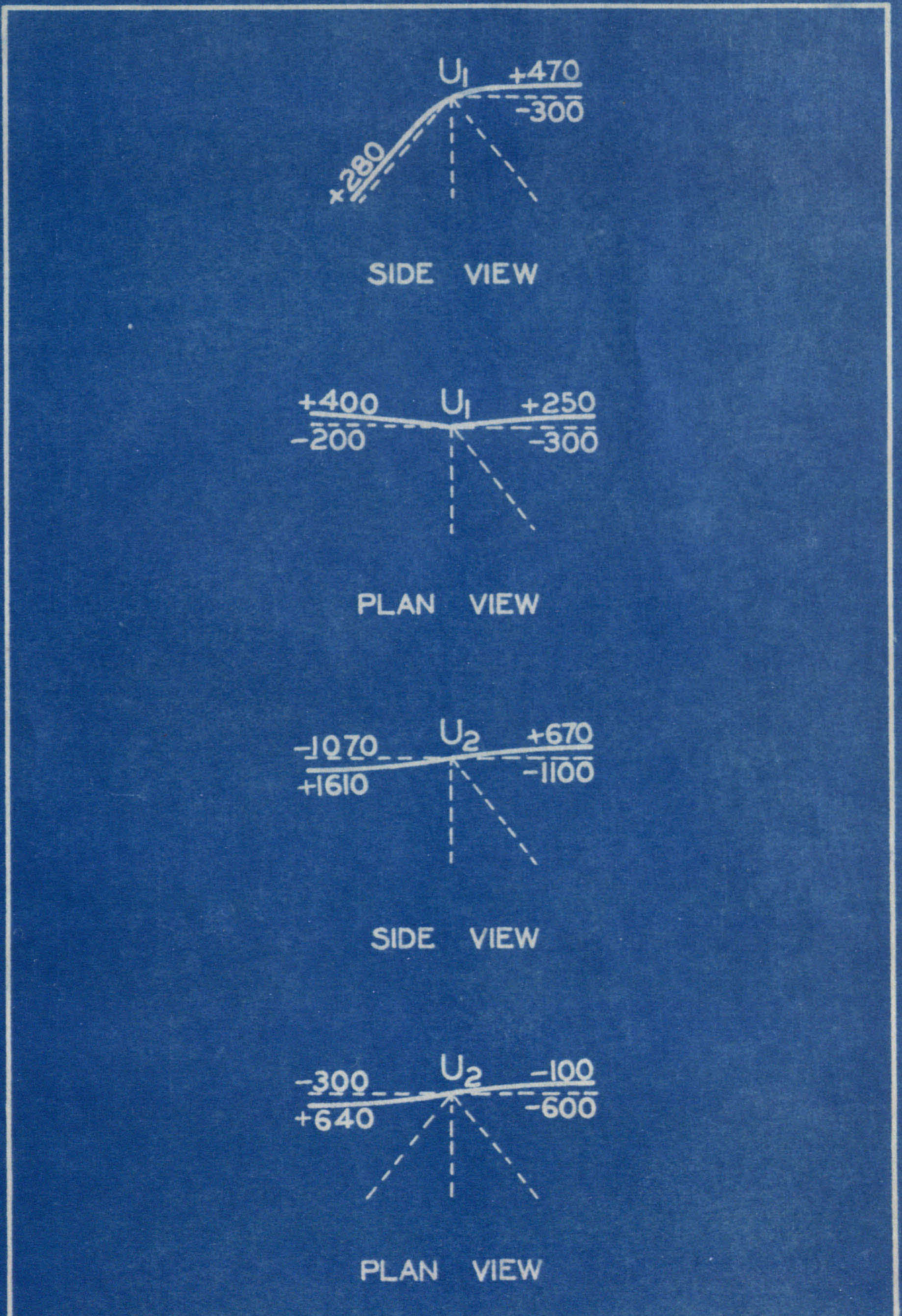



FIG. 19. OBSERVED SECONDARY STRESSES IN END POST AND UPPER CHORD MEMBERS

Form E-5

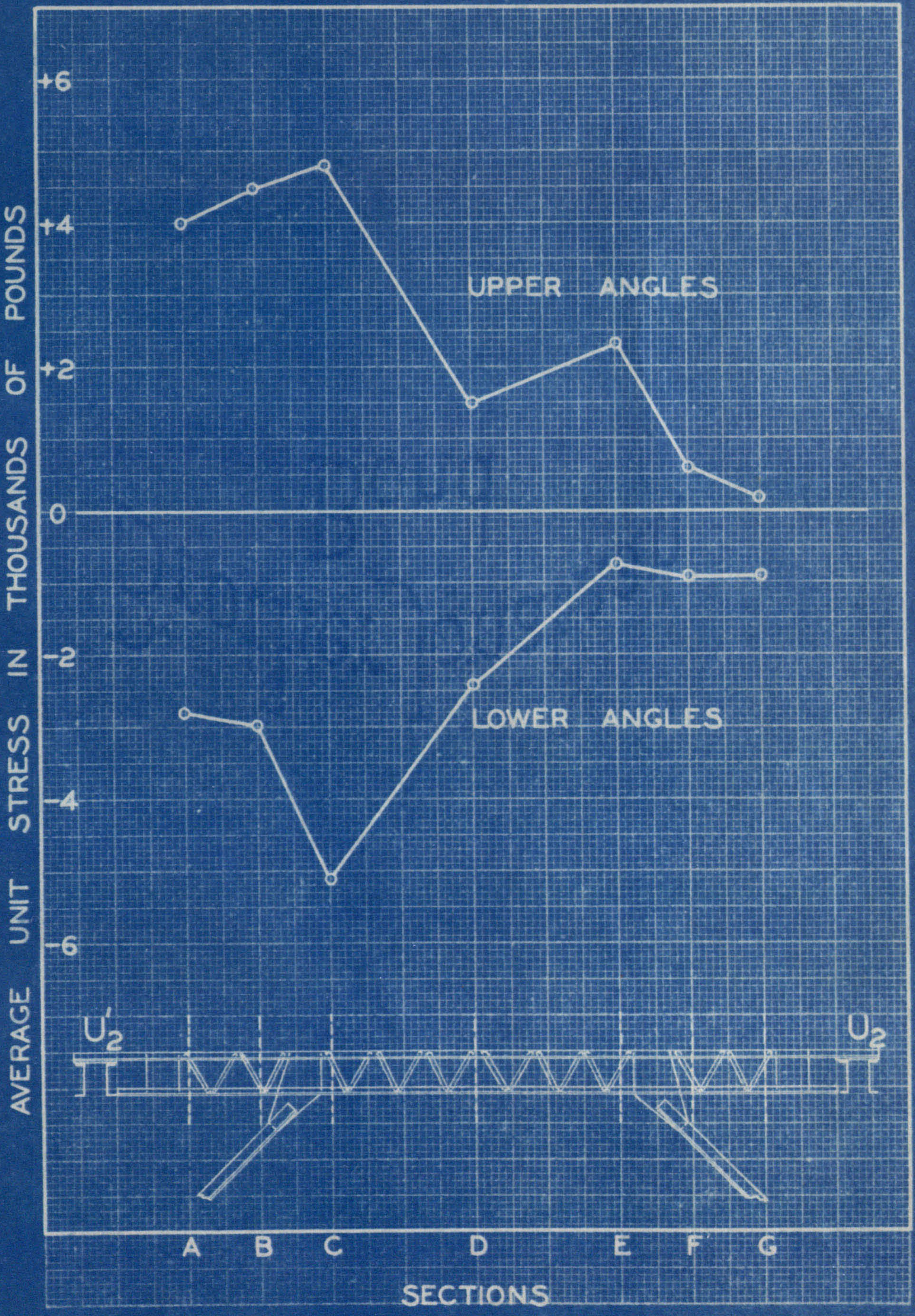



FIG. 20. LONGITUDINAL STRESSES IN UPPER AND LOWER ANGLES OF HEAD STRUT $U_2'U_2$ DUE TO FLOOR LOAD

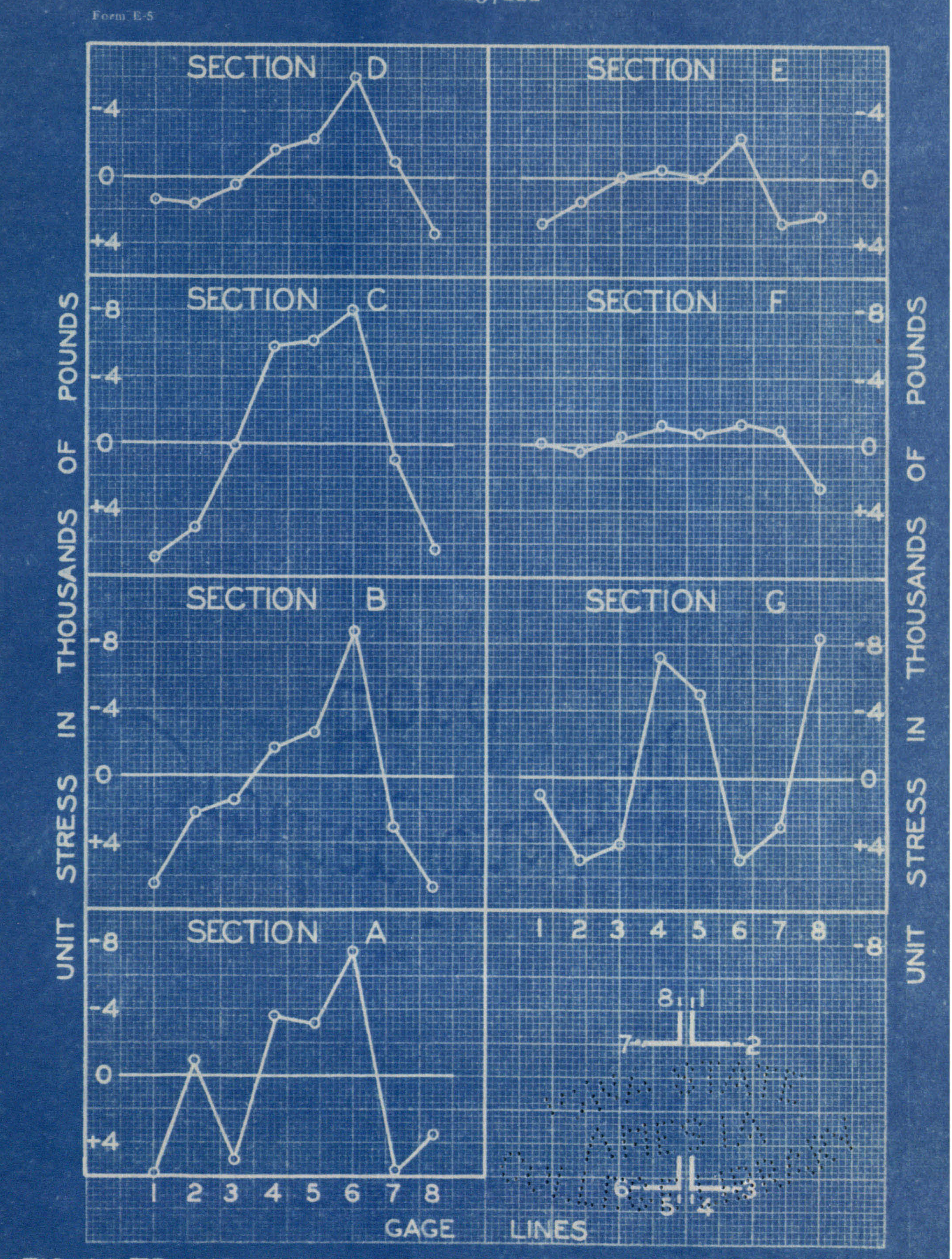



FIG. 21. TRANSVERSE STRESS DISTRIBUTION IN HEAD STRUT $U_2U_2'$ DUE TO FLOOR LOAD

Form E-5

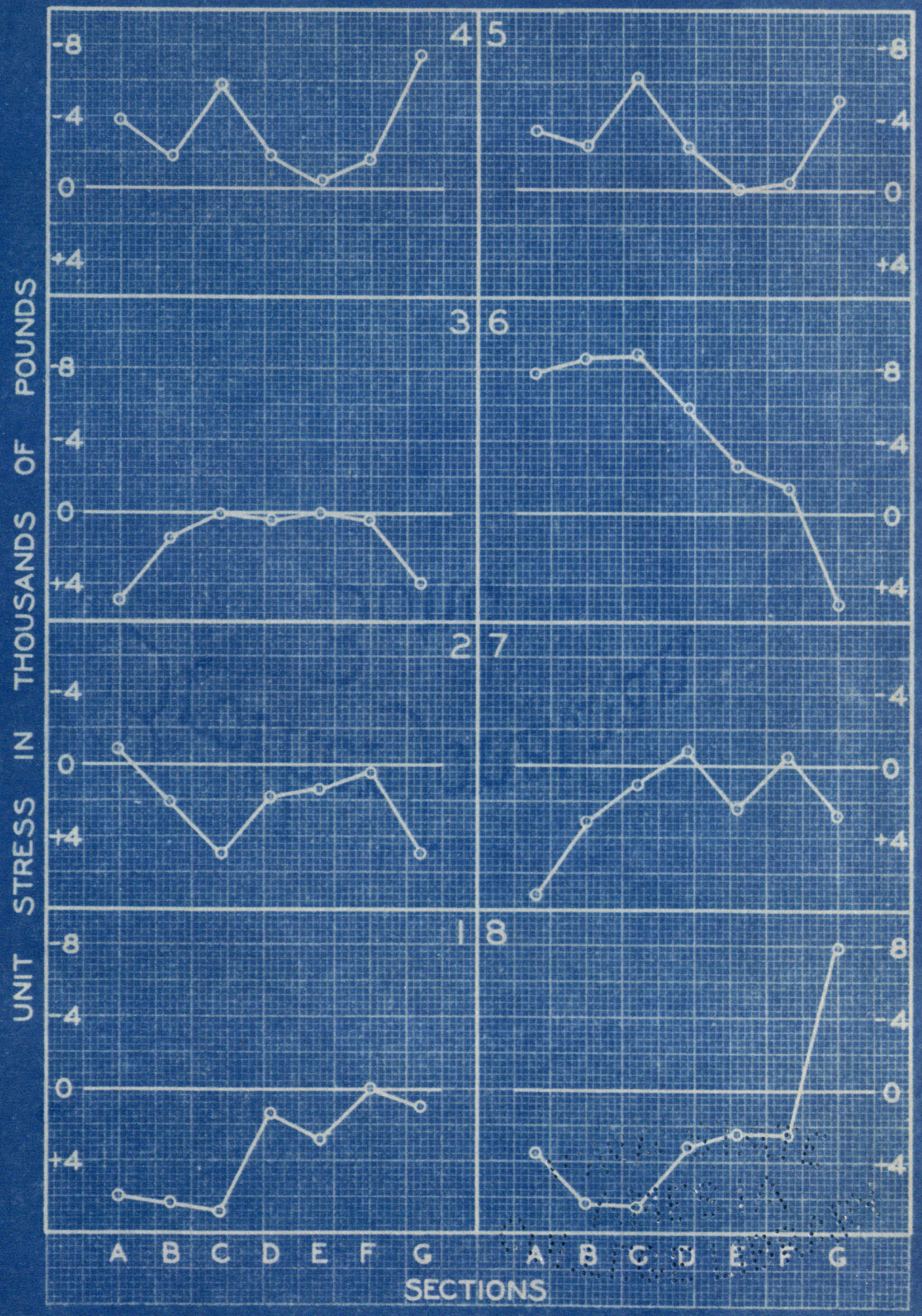



FIG. 22. LONGITUDINAL STRESS DISTRIBUTION IN HEAD STRUT $U_2U_2'$ DUE TO FLOOR LOAD

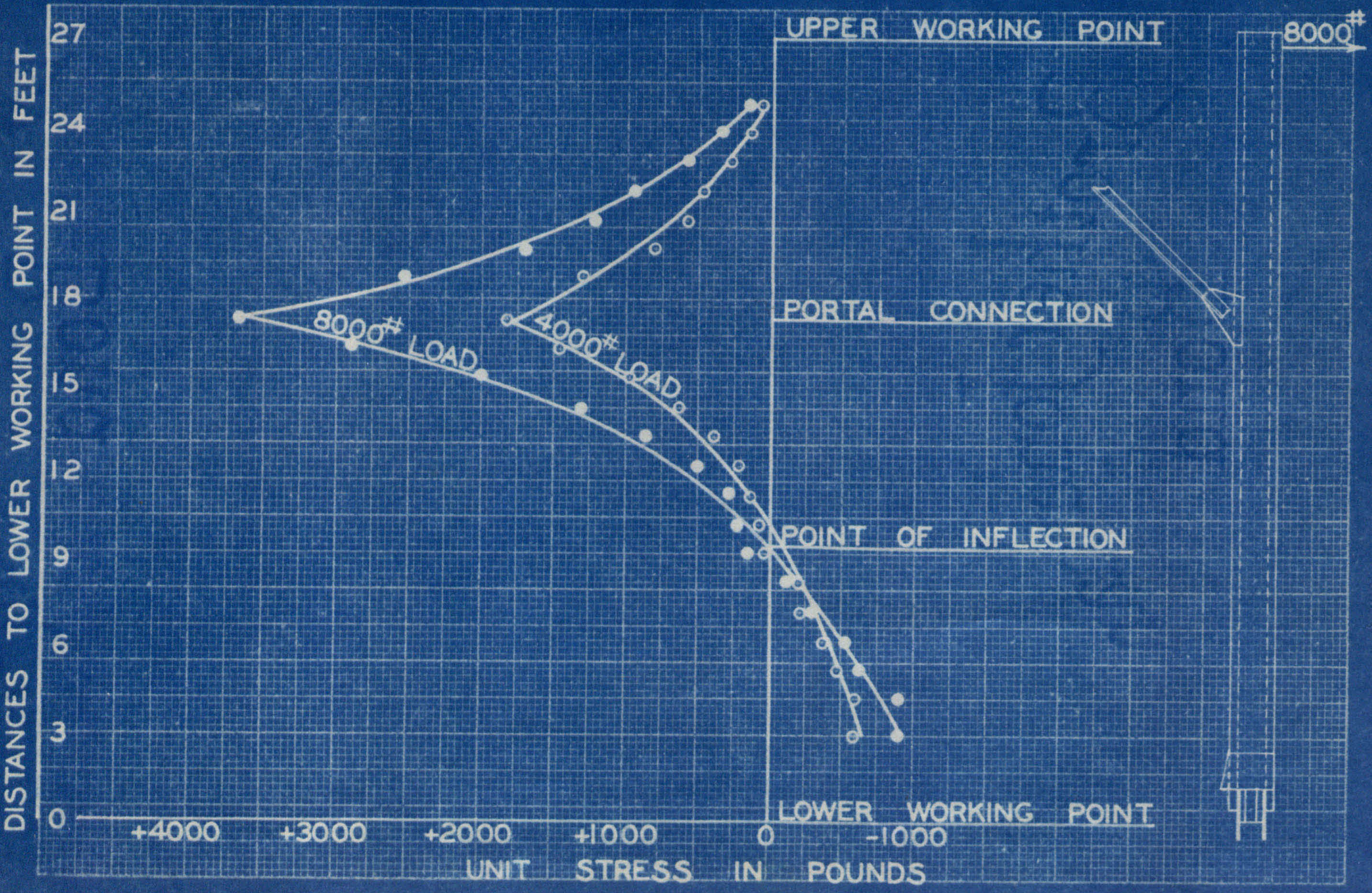



FIG. 23. WIND LOAD STRESS DISTRIBUTION OUTSIDE UPPER FLANGE OF END POST

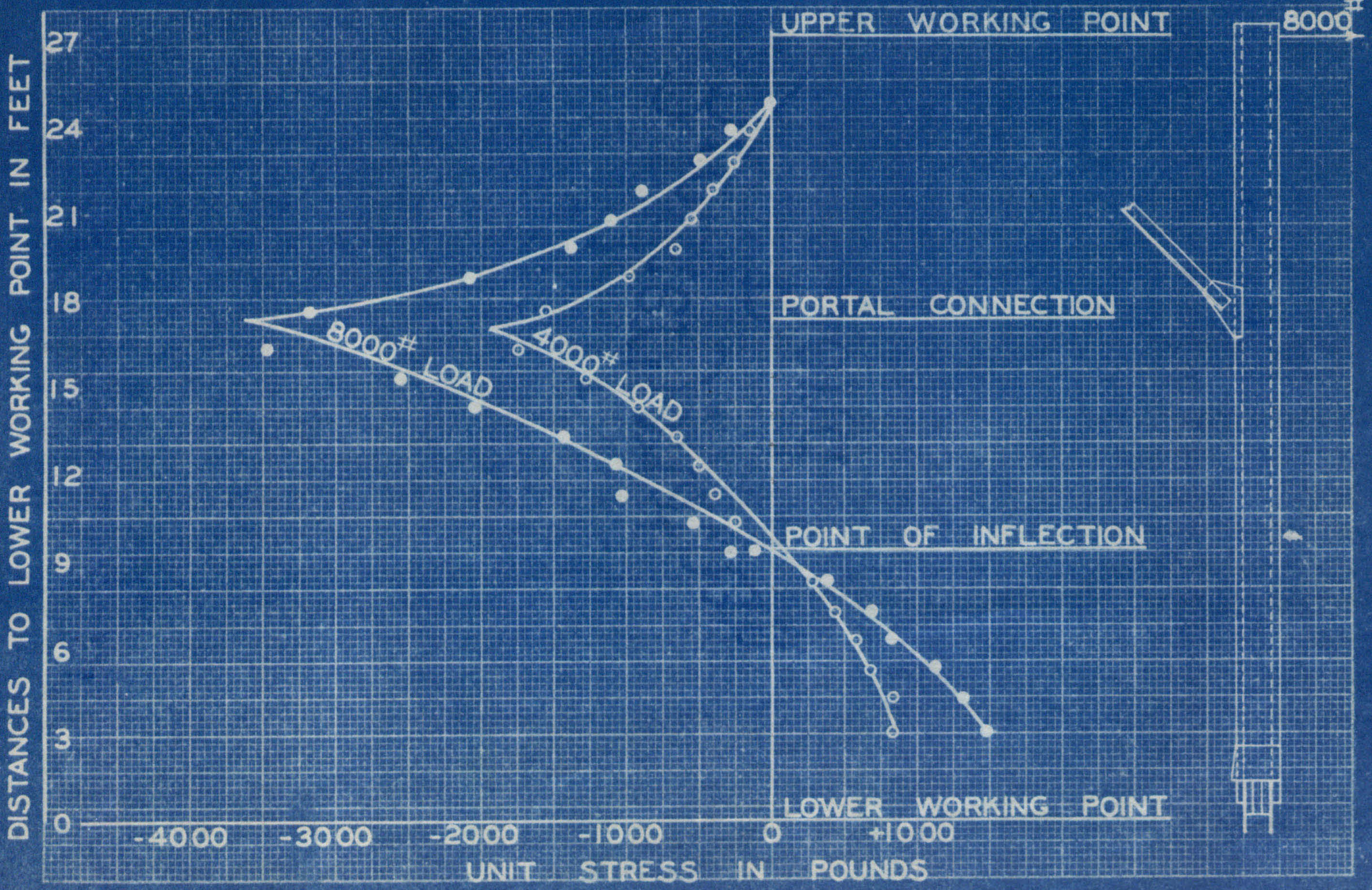
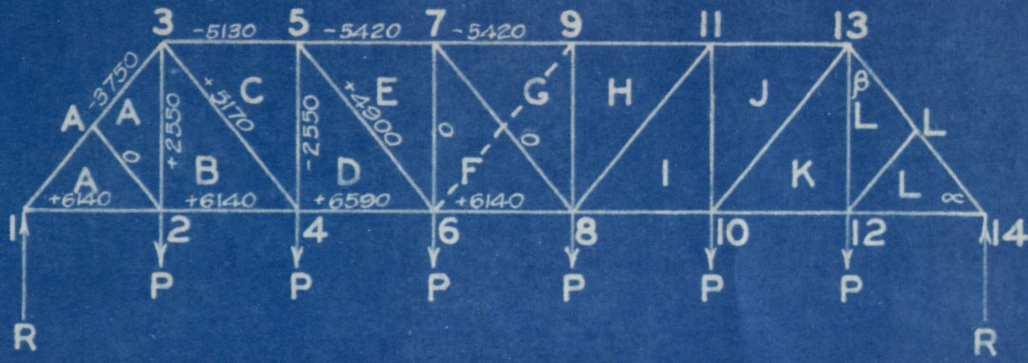
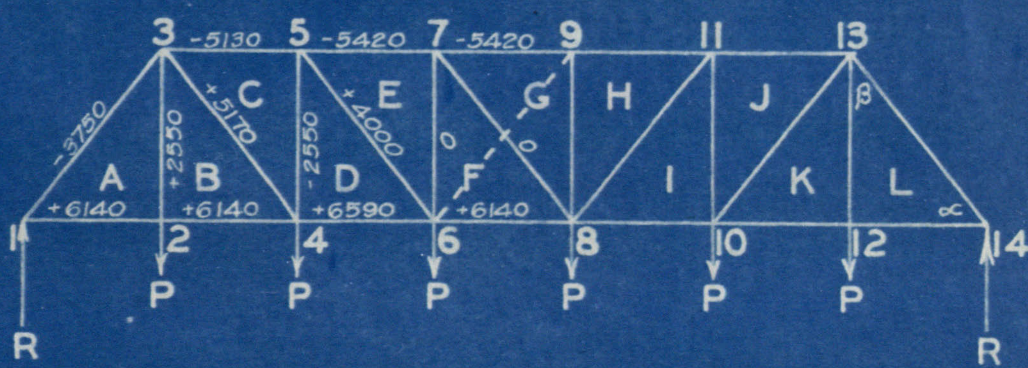



FIG. 24. WIND LOAD STRESS DISTRIBUTION INSIDE UPPER FLANGE OF END POST



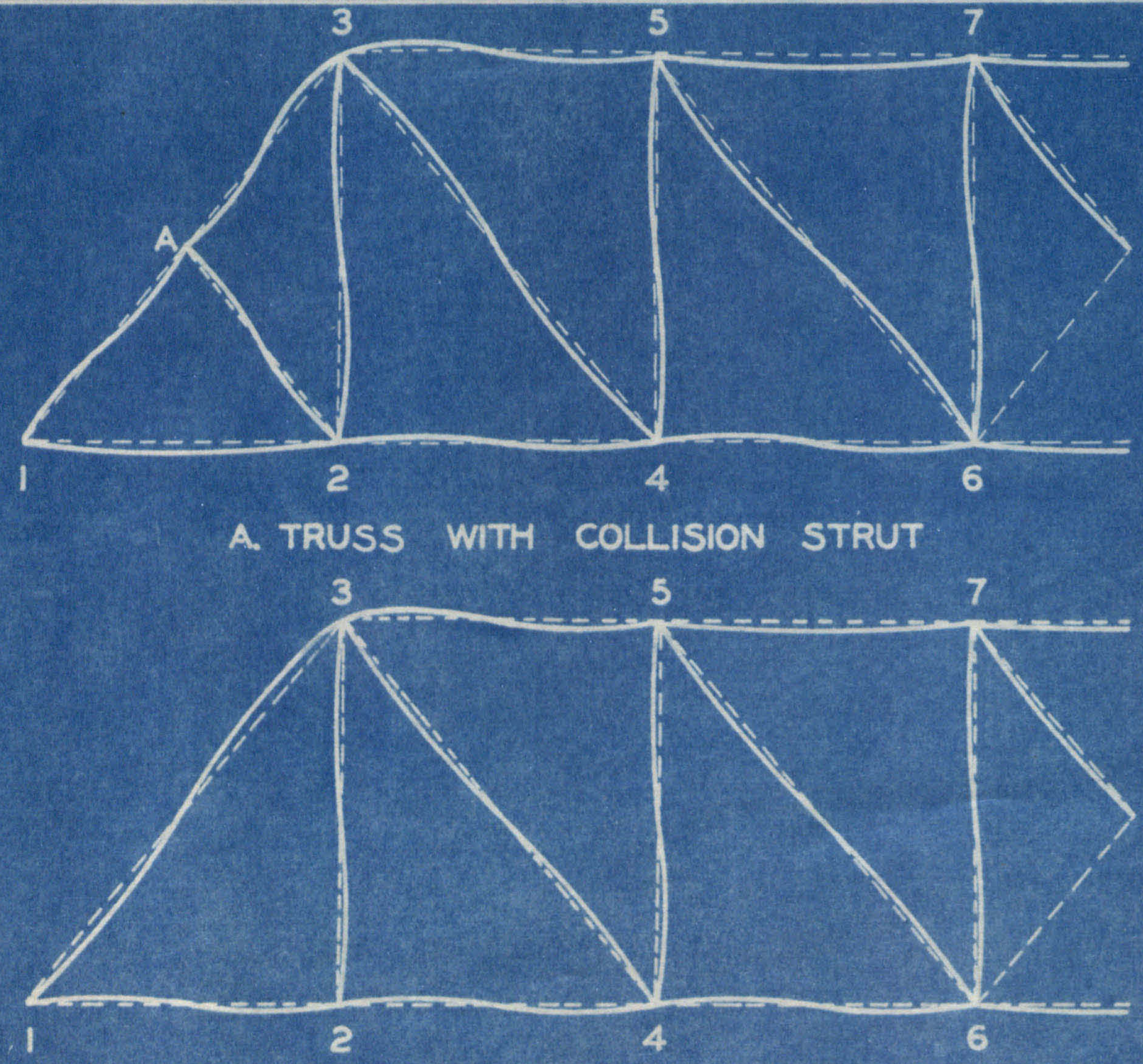

A. TRUSS WITH COLLISION STRUT

COT. $\alpha$ = 0.8165. COT. $\beta$ = 1.2247. P = 17000.# R = 51000.#




B. TRUSS WITHOUT COLLISION STRUT

FIG. 25. COMPUTED FLOOR LOAD STRESS IN TRUSS




B. TRUSS WITHOUT COLLISION STRUT

FIG. 26. DIAGRAMS OF COMPUTED SECONDARY STRESSES